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Larimer Ave. and Atherton Ave. Concrete Arch Bridges, Pittsburgh

The recently completed Larimer Avenue bridge in Pittsburgh, Penn., sets a new record in size for American concrete arches. At the same time it is a notably successful work esthetically, ranking with the Walnut Lane arch, in Philadelphia, in point of appearance. The view on this page, taken about three months ago, brings out quite clearly the well proportioned design, as well as the satisfactory surface finish attained in construction.

At this time several other concrete arch bridges are under construction in the city of Pittsburgh, all of which contain notable features. While not remarkable for size, they are worth special notice, either for type of design, or for nature of material employed, or for esthetic detailing. These bridges are briefly mentioned below, after a description of the larger bridge.

Larimer Avenue Bridge

In the North Highland Ave. district of Pittsburgh, Negley Run gorge is a narrow V-shaped depression 100 to 200 ft. deep in the tableland, running northerly to the Allegheny River. The upper part of this gorge is in two branches. The westerly or smaller of the two is spanned by a concrete arch bridge at Meadow St., built three years ago and fully described in ENGINEERING NEWS of Dec. 1, 1910, p. 583, by N. S. Sprague. About a half mile east of this bridge the other branch of the gorge is crossed by Larimer Ave., a main thoroughfare carrying a street-car line; the car line does not cross the valley at present, but may in the future. A combination wood-and-iron viaduct was built here 1891-1892, consisting of 60-ft. spans on wooden towers. Replacement of this old and inadequate structure by a new bridge was planned at the time the Meadow St. arch was built, and the work was taken in hand after the completion of that structure.

Alternative designs and estimates for a steel arch and a concrete arch for the Larimer Ave. crossing showed no very great difference in cost between the two types. The steel arch if built with stone-masonry piers would, in fact, have cost a little more than the concrete arch. The latter was preferred, and as the available appropriation was sufficient, this type was adopted and built. The resulting structure is very similar to the Meadow St. bridge, but is considerably larger, with a main span of 300 ft. as against the 209-ft. span at Meadow St.

The Larimer Ave. arch is similar to the

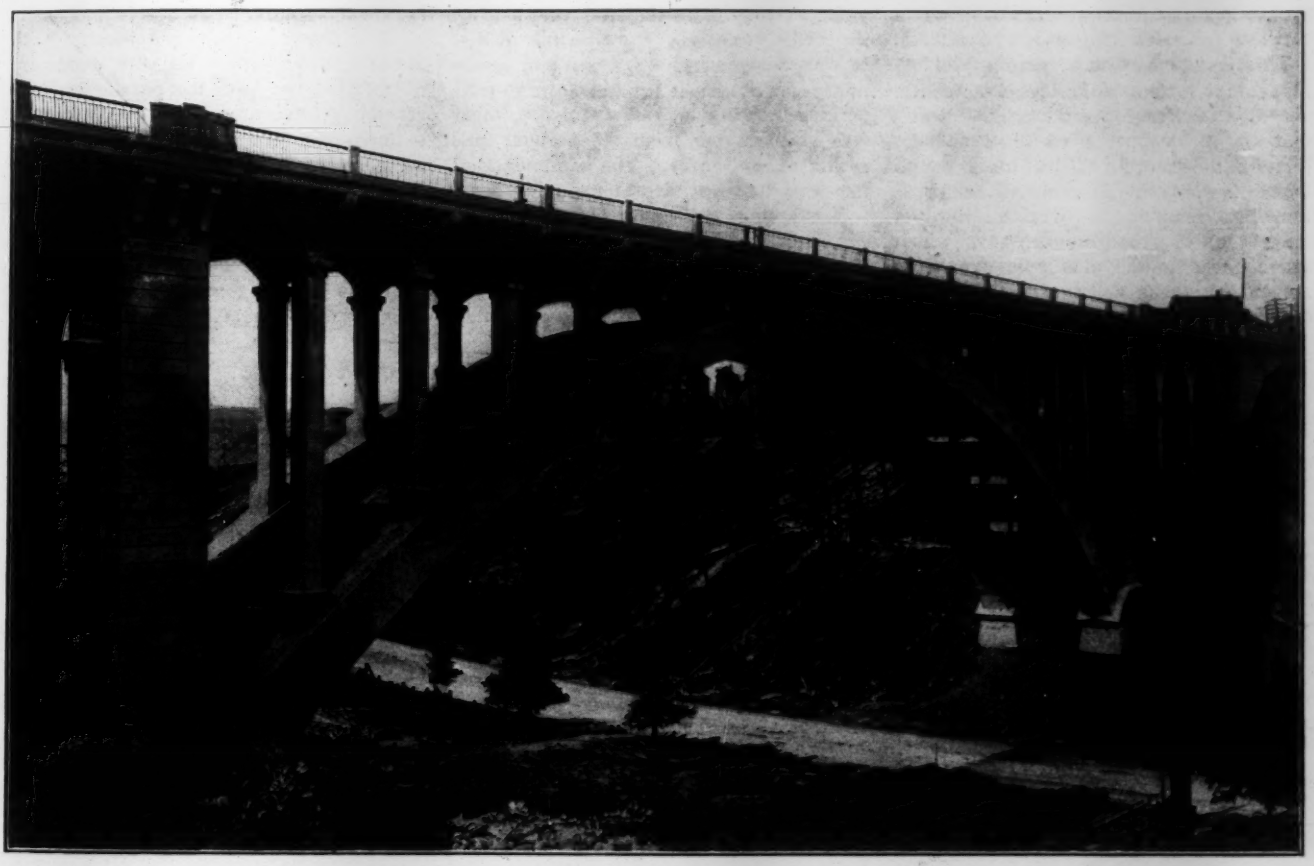


FIG. 1. LARIMER AVENUE ARCH BRIDGE OVER WASHINGTON BOULEVARD, PITTSBURGH, PENN.
(Looking northeast.)

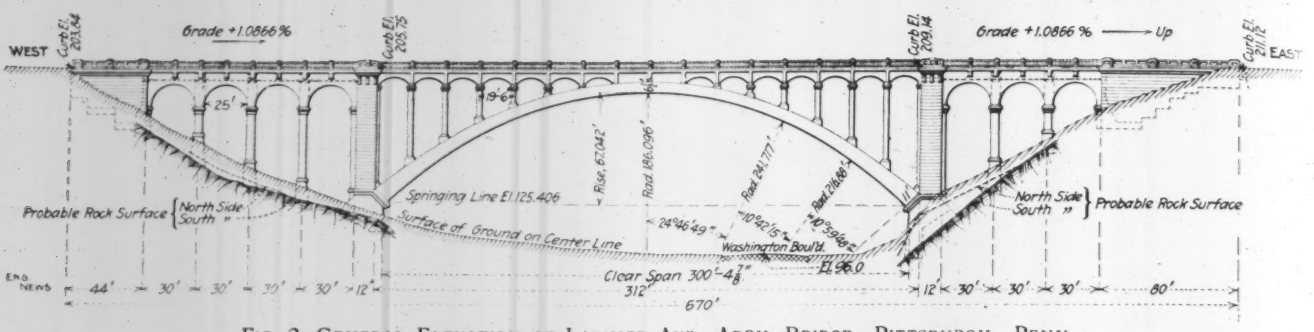


FIG. 2. GENERAL ELEVATION OF LARIMER AVE. ARCH BRIDGE, PITTSBURGH, PENN.

other in having open spandrels, separated arch ribs braced together by struts, and a reinforced-concrete beam-and-slab floor. It differs chiefly in having (1) two ribs, in place of three; (2) rib struts set in the vertical plane instead of normal to the line of the arch; (3) salient skewbacks; and (4) roadway expansion joints between the piers instead of at the piers.

This last item, whose location and arrangement is shown in Fig. 4 herewith, was an afterthought, as the original drawings for the bridge called for expansion joints at the main piers. But it was found that in the Meadow St. arch this construction had resulted in overstressing the longest of the roadway columns, which are exposed to the maximum expansion movement; these columns showed shear cracks, evidently produced by excessive edge of skewbacks to bottom of crown is 67 ft. The springing line is about 30 ft. above the valley, so that the total height from the deck of the bridge to the lowest point of the valley on Washington Boulevard is about 110 ft.

The deck of the bridge rises on a grade of 1.086% to the east (the side of the Brilliant cut-off, Pennsylvania R.R.). It was designed to be a straight grade under maximum load and lowest temperature conditions, and sufficient camber was given to the forms to produce this result (see Fig. 7). The abutments of the arch, however, are at the same level.

The arch is five-centered, approximating parabolic form. The general elevation drawing, Fig. 2, gives particulars. It has two parallel arch ribs uniformly 8 ft. wide, spaced 30 ft. center to center, floor slab between. There is a stringer at each curb and two intermediate stringers, besides a fascia stringer at the outer edge of the sidewalk; the curb stringer has arched lower edge, for appearance.

The bridge was designed for city street loading, with provision for a possible car line in the future. The design was made by the elastic method, graphically, with slight approximation for the departure of form of rib from the parabolic. The reaction-intersection locus was taken as a horizontal straight line; the abutment intersections were found graphically, for parabolic rib form, and the reaction tangent lines were drawn from them. No tension will occur in the rib under any combination of conditions. The compression was limited generally to 600 lb. per sq.in., but in the arch it was allowed to

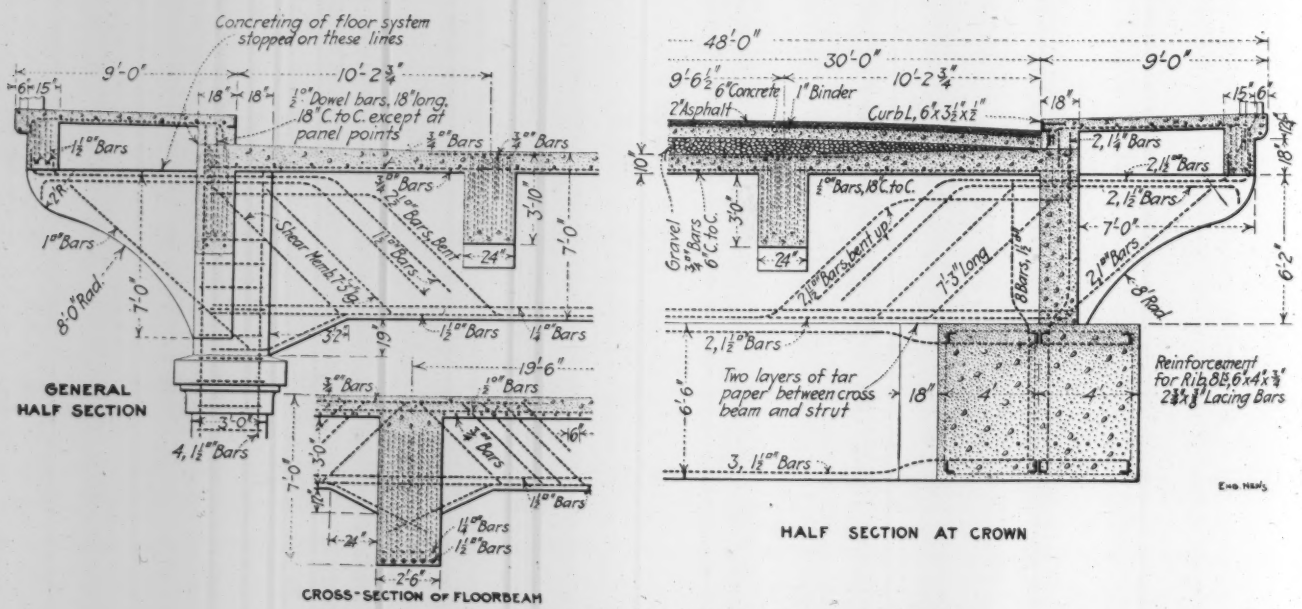

Concreting of floor system stopped on these lines
GENERAL HALF SECTION
CROSS-SECTION OF FLOORBEAM
HALF SECTION AT CROWN
Two layers of tar paper between cross beam and strut
Reinforcement for Rib, 8 Bars, 6"x4"x⅜" 2⅜"x⅜" Lacing Bars

FIG. 3. CROSS-SECTION OF DECK, LARIMER AVE. ARCH BRIDGE

bending under the longitudinal expansion movements. On this showing, the expansion joints in the main-span floor of the Larimer Ave. arch were shifted to points over the haunches.

MAIN ARCH—The arch has a calculated or center-to-center span of 312 ft. Its clear span is 300 ft. 4⅞ in. face to face of abutments. The center-to-center rise is 70 ft., and the clear rise from lower their center lines thus coming under the curbs of the roadway. They are 6½ ft. deep at the crown and 11 ft. deep at the skewbacks. The spandrels are articulated, consisting of individual posts, one on each arch rib, at 19½-ft. panel distance. They carry a reinforced-concrete floor, comprising 30x84-in. floorbeams spaced 19½ ft., and a system of 24x46-in. stringers, with 10-in. reinforced-concrete rise to 750 lb. per sq.in. under the worst temperature combinations. The maximum temperature range was taken at 60° F.

REINFORCEMENT — The reinforcement of the slab and beam work of the bridge calls for no special remarks except to say that deformed rods were used everywhere (specified hot-rolled).

The reinforcement of the main rib, however, employs large-sized steel angles

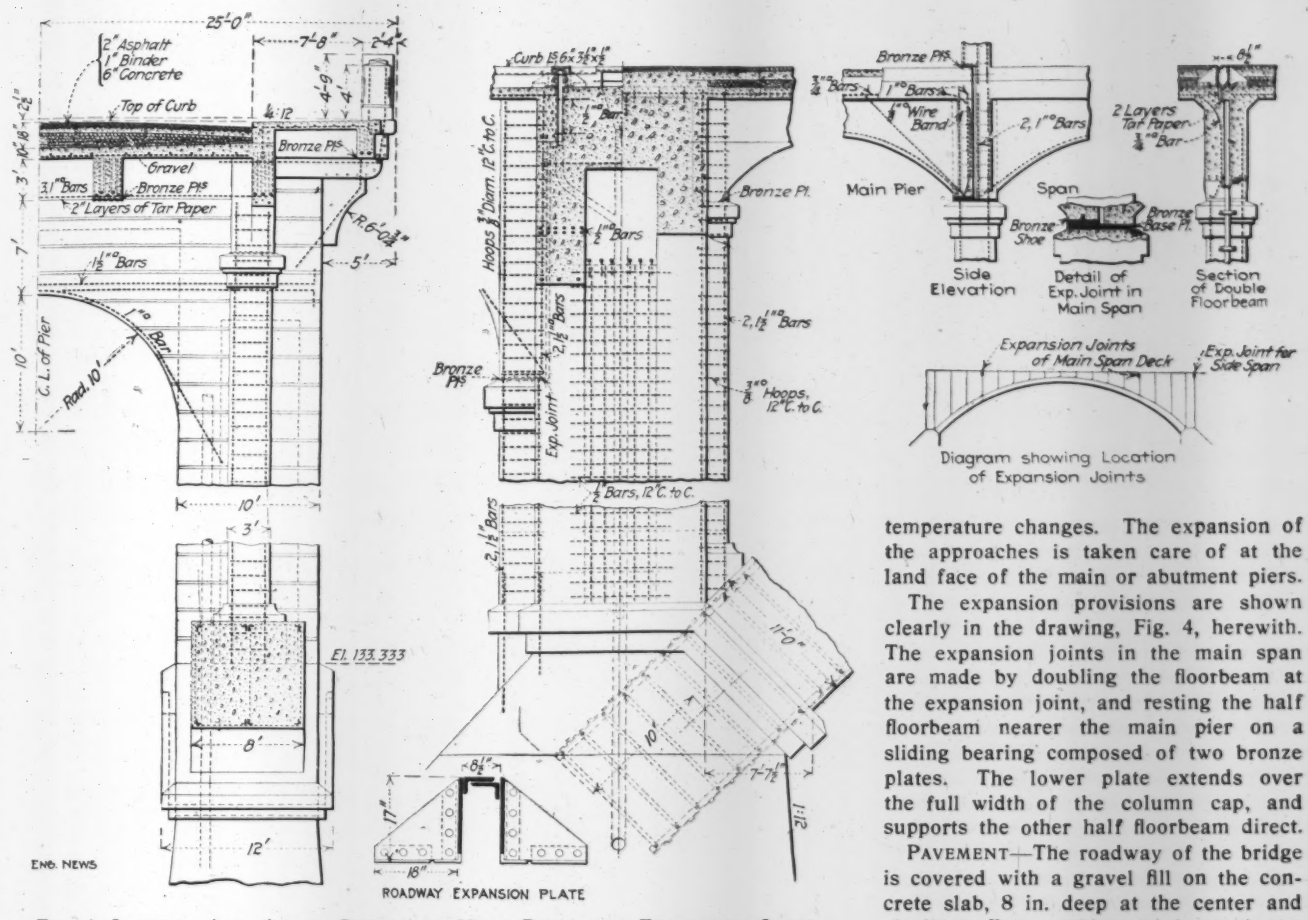


FIG. 4. LARIMER AVE. ARCH: DETAIL AT MAIN PIER, AND EXPANSION JOINT IN FLOOR

extensively, each rib having a longitudinal framework of 8 angles 6x4x¾ in., forming a deep lattice-girder comprising a central I-section girder of four angles and at each corner of the rib section a single angle, the whole well connected by transverse flat bars (no diagonal latticing). Where the rib angles are spliced, the joint is made strong enough to transmit the full stress of the angles. These splices are placed in the key blocks of the arch, and are field splices, made with turned bolts in drilled holes. The total cross-section of reinforcement in each rib is 55.52 sq.in., constant from end to end of the arch.

The rib reinforcement is supplemented at the points where the columns carrying the roadway rest upon it. The special reinforcement here comprises a group of bent stirrup bars in the rib, vertical bars (prolongations of the column bars) passing nearly through the rib, and a system of horizontal bars to tie the columns into the arch. This is fully shown in the detail sketch Fig. 5.

EXPANSION JOINTS—In the Meadow St. arch the expansion of the roadway was taken up by expansion joints at the piers. On that bridge the longest columns of the roadway have developed cracks which indicate that the expansion movements here are excessive. On account of this experience the expansion joints of the roadway in the Larimer Ave. bridge were located at the third panel-point out from the main piers. The sidewalk and its supporting fascia girder, however, have an expansion joint in each panel, to allow for the more rapid response of these thinner parts of the structure to temperature changes. The expansion of the approaches is taken care of at the land face of the main or abutment piers.

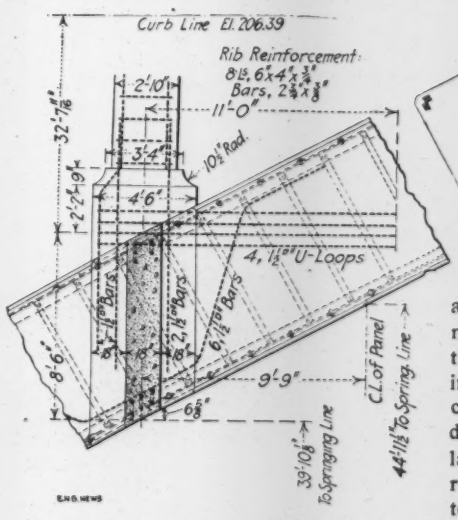
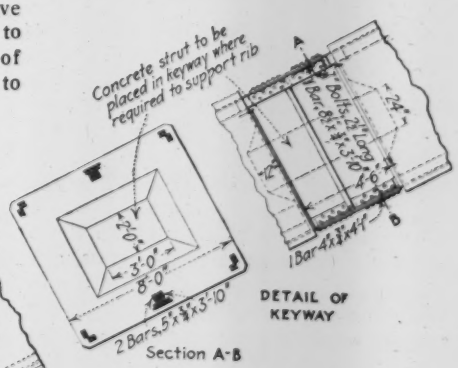


FIG. 5. LARIMER AVE. ARCH: RIB AT PANEL-POINT 3

The expansion provisions are shown clearly in the drawing, Fig. 4, herewith. The expansion joints in the main span are made by doubling the floorbeam at the expansion joint, and resting the half floorbeam nearer the main pier on a sliding bearing composed of two bronze plates. The lower plate extends over the full width of the column cap, and supports the other half floorbeam direct.

PAVEMENT—The roadway of the bridge is covered with a gravel fill on the concrete slab, 8 in. deep at the center and shading off to nothing at the curbs; a 6-in. layer of 1:3:6 concrete over this; then a 1-in. binder and a 2-in. sheet of asphalt, bringing the final surface at the middle of the roadway width 1 in. below curb level.

APPROACHES—The approaches on either side of the main span consist of what appear to be semicircular arches, spanning 30 ft. center to center and 25 ft. in the clear. The actual framing, however, involves longitudinal girders of reinforced concrete with the curb girder carried down to form a diaphragm of semicircular arch contour, this diaphragm having reinforcement both at the line of the bottom of its straight girder portion and along the intradosal line of the arch face. The deck construction is in reinforced-



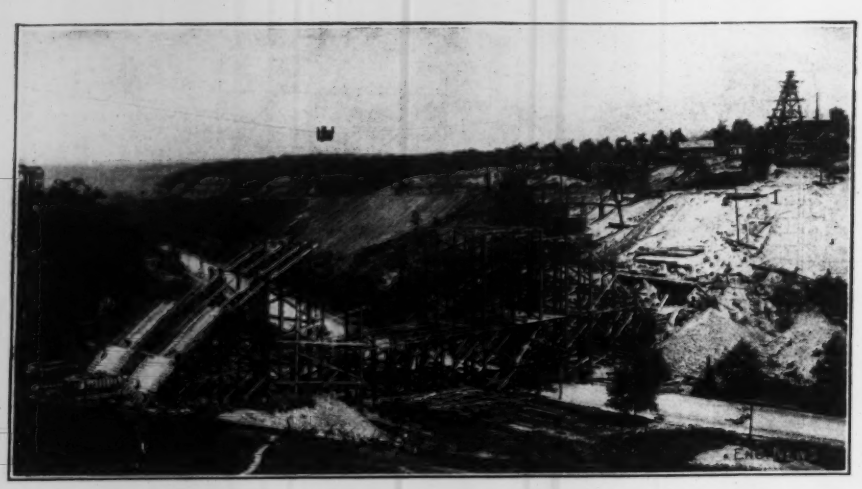


FIG. 6. BUILDING THE CENTERING FOR THE LARIMER AVE. ARCH

concrete slab arrangement as over the main span.

FORMS AND CONCRETING—The order of concreting was as follows: The two ribs were concreted simultaneously, each being formed in five different sections on either half arch. The sections were placed first in the prescribed order, and later the keys between them were concreted to make the closure. At the roadway columns the rib form included also the base portion of the column, so that a level seat for the column resulted. The ...umns were next cast, up to the top of the cap. Then the floor forms were built complete from end to end, and the whole aggregation of floorbeams, stringers, and slabs, together with the sidewalk brackets, concreted in one consecutive series of operations. The sidewalk slab with the curb and the sidewalk fascia girder was cast later; the separation of this part from the main roadway structure was maintained on account of the expansion arrangements.

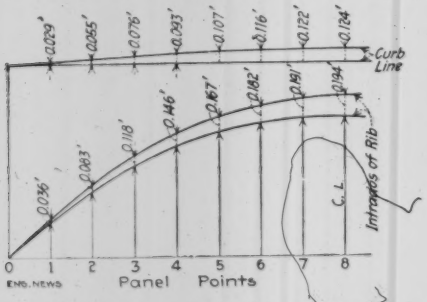


FIG. 7. LARIMER AVE. ARCH: CAMBER DIAGRAM FOR MAIN SPAN

(The cambers given do not include false-work camber.)

For the rib concreting the centering for the two ribs was built complete and the rib reinforcing then placed continuous from end to end. The form walls for the sides of the rib were placed in sections, and closed off at the ends of the sections by bulkheads. The arrangement of the individual sections and the prescribed order of concreting are given by the diagram Fig. 9. The longitudinal reinforcing necessarily passed through the bulkhead, and remained exposed here until the key sections were concreted. The bulkheads were provided with large projections on their inner faces in the form of frustums of pyramids to form pockets in the ends of the rib sections in order to give a better engagement with the concrete of the key sections. These projections were made of such shape that air pockets were avoided, the slope of the inclined faces being made steep enough to avoid any reverse slope.

Each section was concreted in a single day. The top form of the section was laid up as the concreting progressed. Concrete was mixed at a mixing plant at the top of the slope on the east side of the valley and brought to the work in buckets by the cableway which may be seen in the construction views.

When the ribs were concreted and had set, the centers were struck, forms were erected for the columns to top of cap and these were then cast. At this stage the work was interrupted by the winter of 1911-12; the erection of floor forms and concreting of floor were done in the spring of 1912. In the floor concreting, the joints between successive days' work were made transversely across slab and stringers midway between floor-beams.

MATERIAL—The structural part of the bridge consists of 1:2:4 concrete, using "Universal" cement, river sand, and 1-in. Ligonier limestone. The sidewalk slab was made with 1-in. Ligonier stone in the body, and screenings of the same stone for the wearing surface, troweled hard and then floated with rough wood floats to roughen the surface. In finishing this wearing coat, it was found that the aggregate was too fine to work well under the floats, and a proportion of ¼- to ½-in. stone had to be mixed in so as to make the concrete coarser and less mobile.

All concrete was mixed quite wet, very mushy.

FIG. 9. LARIMER AVE. ARCH: SKETCH SHOWING ORDER OF CONCRETING SECTIONS OF ARCH RIBS

(Section E placed monolithic with main piers. Other sections concreted in order A, B, C, D.)

Surface Finish—The Larimer Ave. arch is noteworthy for the very successful surface finish attained upon it. The main surfaces everywhere were made by spading back the concrete, and shortly after completion were bush-hammered. A surface of excellent texture and very uniform color tone was thus secured. The

FIG. 8. LARIMER AVE. ARCH CENTERING, WITH RIB REINFORCING IN PLACE, AND FORMS FOR PARTS OF PIERS AND RIBS

(West main pier, looking northeast, Aug. 21, 1911).

bush-hammering operation, a very extensive undertaking inasmuch as the whole bridge was involved, added about 3% to the cost of the structure.

The bush-hammered finish was used for all the main faces, but the bases and caps of columns, skewback coping, and a few other elements of the structure, were rubbed with a carborundum brick and then painted with a very thin cement-sand grout. The resulting texture and color differ slightly from that of the main faces, giving an effective relief and contrast.

A further departure from the general bush-hammering treatment was used in the posts of the roadway balustrade, in order to secure more decorative effects. The balustrade consists of steel railing between concrete posts or piers, connected by bronze fittings. These piers are recessed and paneled on their inner and outer faces. The general surface of the post was troweled immediately after stripping the form, and a smooth, hard surface obtained. The panels, however, were treated differently. In placing the concrete, the stone here was not spaded back from the face, but on the contrary was allowed to come into contact with the form as thoroughly as possible, and after removal of the form the panels were bush-hammered. This produced a surface showing the stone exposed in an irregular white mosaic in the gray or bluish cement background. The effect of this finish is in every way pleasing and ornamental.

ORNAMENTATION — The ornamentation of the bridge lies wholly in the proportioning and finish of its parts and the detailing of copings, railings, etc. Practically the only essentially ornamental item will be four bronze and granite candelabra, one to be set over each of the four main piers. These candelabra were designed by the architect of the city's Bureau of Construction. They are 8 ft. 8 in. high above the top of the balustrade. The bronze shaft carries a ground-glass ball containing incandescent lamps.

On the east side of the bridge, between the arch abutment piers and in view of passers-by on the Washington Boulevard below, a commemorative tablet is to be placed. This will consist of a bronze inscription plate bolted to a concrete wall erected between the abutment piers.

GENERAL—The bridge was designed and its construction was supervised by the Bureau of Construction of the Department of Public Works of Pittsburgh (J. G. Armstrong, Director of the department; N. S. Sprague, Superintendent of the bureau). Willis Whited was division engineer during the design, with T. J. Wilkerson as designing engineer. latter succeeded Mr. Whited and was in charge during construction. The contract was let to the John F. Casey Co., of Pittsburgh, for the price of $140,948. Extras, and variations of actual from estimated quantities, will bring the total cost of the bridge to from $145,000 to $150,000. O. Stange was superintendent of construction for the contractor.

FIG. 10. CONCRETING RIB SECTIONS OF THE LARIMER AVE. ARCH (SEPT. 6, 1911)

The contract was let on Apr. 22, 1911, and work was begun early in May. The contract date of completion was July 1, 1912, but various incidents delayed completion until August, 1912, although the formal opening took place on July 10. In the 1911 season the pier and abutment foundations were built, the main arch concreted, the main piers constructed, and the columns on the main arch placed. The west approach also was constructed during this season. In 1912 the entire deck was formed and concreted, the east approach built and the surface-finish work done.

The Larimer Ave. bridge contains 8525 cu.yd. structural concrete, 892,540 lb. reinforcing steel, and 2233 sq.yd. pavement. The cost per unit is $208.50 per lin.ft. bridge, or $4.17 per sq.yd. ground area.

FIG. 11. LARIMER AVE. ARCH: FORMS FOR FLOOR FRAMING AND SLAB (MAR. 18, 1912)

Atherton Ave. Bridges

Two concrete arch bridges are being built to carry Atherton Ave. over the break caused by the lines of the Pittsburgh Junction R.R. and the Pennsylvania R.R. crossing through the Pittsburgh peninsula. The new street connection will provide an important traffic route from Grant Boulevard to the East Liberty section.

The two crossings are a quarter mile apart. The westerly one, the Pittsburgh Junction R.R. crossing, is a single arch span of large size, and therefore its design embodies separation of the arch barrel into rings, open spandrels, and beam-and-slab floor construction. The Pennsylvania R.R. crossing has to do with a wider and shallower valley, for which a series of short spans was suitable. This bridge has full-barrel arches, solid spandrel walls, and filling over the arch. The two structures involve about equal quan-

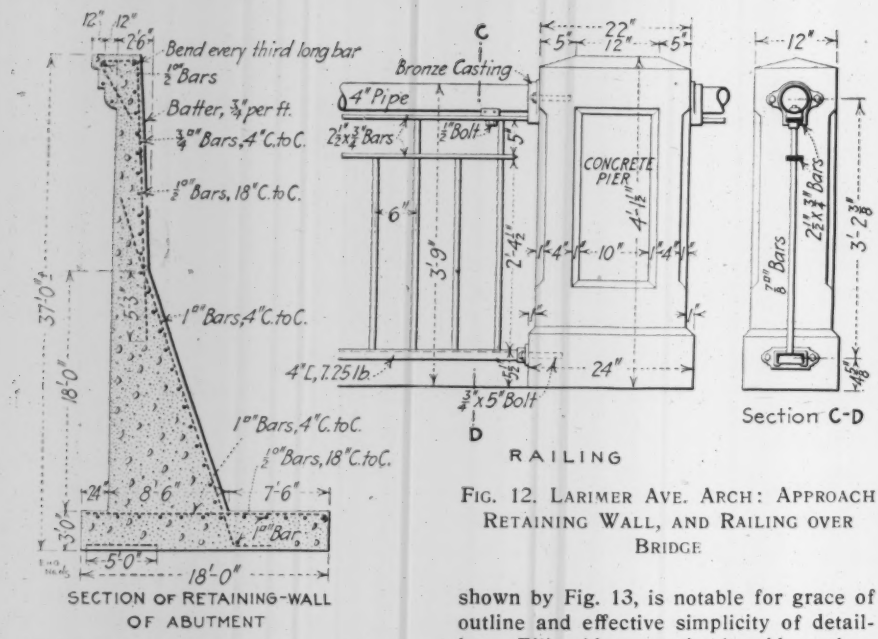


FIG. 12. LARIMER AVE. ARCH: APPROACH RETAINING WALL, AND RAILING OVER BRIDGE

tities of material, however, and cost about the same—a trifle over $90,000.

PITTSBURGH JUNCTION R.R. BRIDGE

The design of the Pittsburgh Junction R.R. bridge, with its 168½ ft. large span and its 45-ft. and 60-ft. flanking spans, shown by Fig. 13, is notable for grace of outline and effective simplicity of detailing. Elliptoid curves in the side arches, and a related effect in the large arch secured by the treatment of the ends, gives the structure a bold and full effect. Ribs, posts and quoins are brought out by rustication or false-joints, as against smooth surfaces in the Larimer Ave. bridge.

The structural design of this arch followed generally the methods applied to the Larimer Ave. arch. The two detail sections included in Fig. 13 indicate the arrangement of the rib reinforcement and the floor construction.

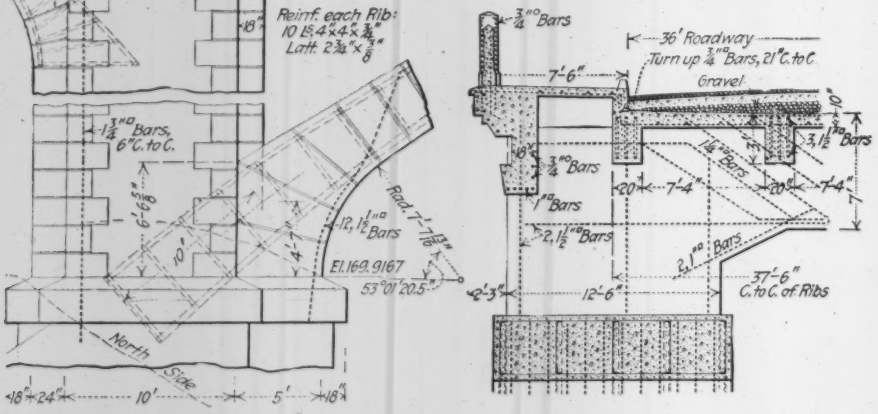
The large arch has a figured span of 177 ft. and a figured rise of 29¾ ft. It consists of two ribs of uniform width, as in the case of the Larimer Ave. arch, but they are wider, and cross-struts are therefore dispensed with (each rib 14 ft. wide, against 8 ft. in Larimer Ave.). The crown thickness is 42 in., the thickness at springing line 68 in. The ribs are 37½ ft. apart c. to c., which brings them nearly but not quite central under the curbs. The sidewalks do not overhang as in the Larimer Ave. bridge.

The deck is 60 ft. wide at piers and abutments, and over the spans is 54½ ft. wide to sidewalks, with a 36-ft. roadway. It is a reinforced-concrete slab-and-beam floor on columns 3x12½ ft. resting on the arch ribs. These wide columns, in conjunction with the floor-beams, act as portal frames to conjoin the lateral stiffness of rib and floor.

The spandrel arches disappear near the middle of the span, and a solid spandrel wall replaces them. Pilasters mark the positions corresponding to the columns.

The division of the main arch into two ribs is carried through the main piers, these being pierced by a 19-ft. opening, leaving an 18½-ft. pier on either side. Thus, standing under one of the approach arches a clear view may be had to the opposite abutment. The approach arches also are divided into two ribs, though their spandrels are full walls; however, their floorbeams are set close and no stringers are used.

The main arch ribs were concreted in six sections each, with short key spaces between. The short-radius portion near the skewbacks was concreted as part of the main piers. The rib concrete was carried up at each of the columns to form the bottom block of the column and give a level seat for the column body. The floor was concreted in sections, the forms, however, being built continuous and complete. Both rib and floor forms were set for camber, as in the Larimer Ave. bridge, to take up the deflection due to dead-load; this camber was 0.104 ft. at



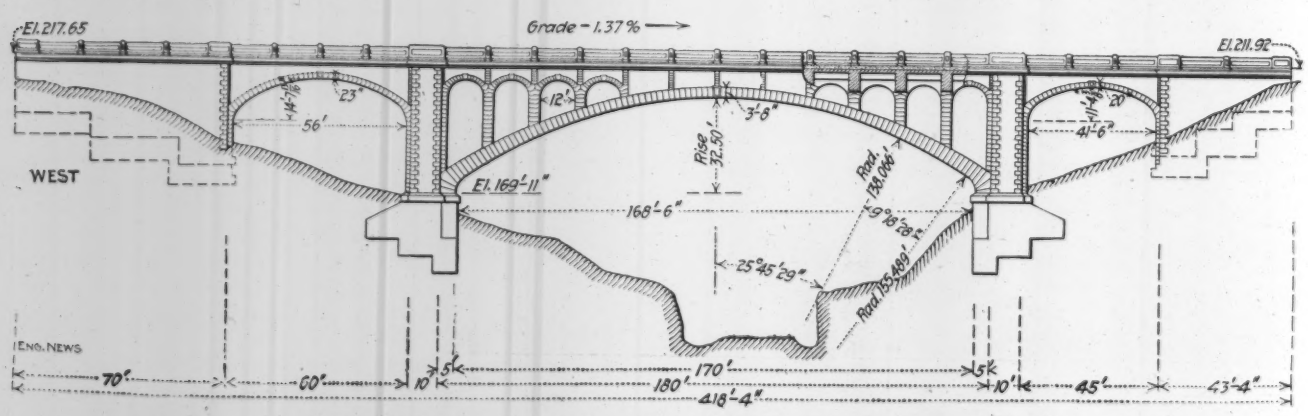


FIG. 13. ATHERTON AVE. BRIDGE OVER PITTSBURGH JUNCTION R.R.

FIG. 14. CENTERING FOR ATHERTON AVE. BRIDGE OVER PITTSBURGH JUNCTION R.R., SEEN LOOKING NORTH

(A 60-ft. Howe truss over the tracks carries the middle part of the centering. The concrete-distributing tower stands at the left.)

FIG. 15. WEST PIERS AND ABUTMENT OF PITTSBURGH JUNCTION RY. BRIDGE, FROM TOP OF MAIN ARCH CENTERING

crown for the ribs and 0.089 ft. for the floor.

The approach arches were concreted in three sections, crown first and haunches afterward. No camber was allowed for.

Expansion joints are built in the floor at the main piers. The stringers of the main span are set in pockets here, on bronze plates. The slab contains a double steel curb as at the expansion points of the Larimer Ave. bridge. On the landward edge of the pier the slab of the approach span is divided from the pier concrete by a construction joint, made by tar paper.

The block-jointed surfaces of the bridges were rubbed and then brushed over with cement wash. The panels between are to be bush-hammered.

The contract for the bridge was let on Aug. 16, 1911, to the Friday Contracting Co., of Pittsburgh, for a price of $91,330. It will be completed this winter. The design was made and detailed by the Bureau of Construction, Department of Public Works.

The bridge contains 7960 cu.yd. structural concrete, 664,000 lb. reinforcing steel, and 1672 sq.yd. paving. The contract price reduced to units is $3.62 per sq.ft. ground area, or $217 per lin.ft. bridge.

PENNSYLVANIA R.R. BRIDGE

The bridge over the valley through which the Pennsylvania R.R.'s four-track line passes is structurally much simpler, though not simpler as a piece of contract work. Fig. 16 shows its general features.

It is a series of three 90-ft. skew arches, 49½ ft. wide, over the arch ring, with solid-fill abutments making up the approaches. The arches have full sheet ring, carrying solid spandrel walls, and the interior is to be filled with granulated blast-furnace slag on which the pavement will be laid. The roadway has a fall of 1% eastward, and the springing-lines of the arches are arranged on the same slope.

The three arches are alike. The rise on the 90-ft. span is 21 ft. The intrados is a circular segment of about 63½ ft. radius, except that an 11-ft. length at each end is of 35 ft. radius. The form is symmetrical about a line normal to the grade, and not symmetrical about the midspan vertical. The barrel of the arch is 30 in. thick at the crown, with rapid increase in thickness toward the piers; at the springing-line vertical the depth of the barrel is 10 ft. 3 in.

The barrel is heavily reinforced with rods at extrados and intrados. At intervals of about 11 ft. along the span a lattice-girder with single-angle flanges extends transversely across the arch. The spandrel walls, reinforced by vertical rods and anchored to the ring by horizontal rods, are tied to the girders by counterforts of triangular outline, containing diagonal rods fastened to the upper flange-angle of the girders.

The esthetic treatment of the structure is simple. Semi-circular pier-nosings are used, being set normal to the spandrel walls and not in line of the piers. The spandrels have recessed panels, and the solid concrete parapet also has recessed paneling. The panels are bush-hammered.

The bridge is being built by the Cranford Construction Co., under a contract let Feb. 24, 1912, for $93,128. A centering of steel arch trusses is used for carrying the arch lagging. Concrete is chuted from a distributing tower located to one side of the bridge on the flat westerly slope.

This bridge will contain about 7300 cu.yd. structural concrete (i.e., excluding paving concrete), 419,000 lb. reinforcing steel, and 1500 sq.yd. paving. Its contract cost amounts to $246 per lin.ft., or $4.13 per sq.ft. area covered.

BRICKBAT CONCRETE

A year ago the Bureau of Construction made some trials to produce a concrete differing in color and texture from ordinary concrete. The bush-hammered panels of the Larimer Ave. bridge, giving a rough mosaic effect, have been mentioned. The effect of a similar treatment when broken brick or terra-cotta was used as aggregate was tried, and attractive results secured. The bush-hammering exposed bright reddish surfaces of

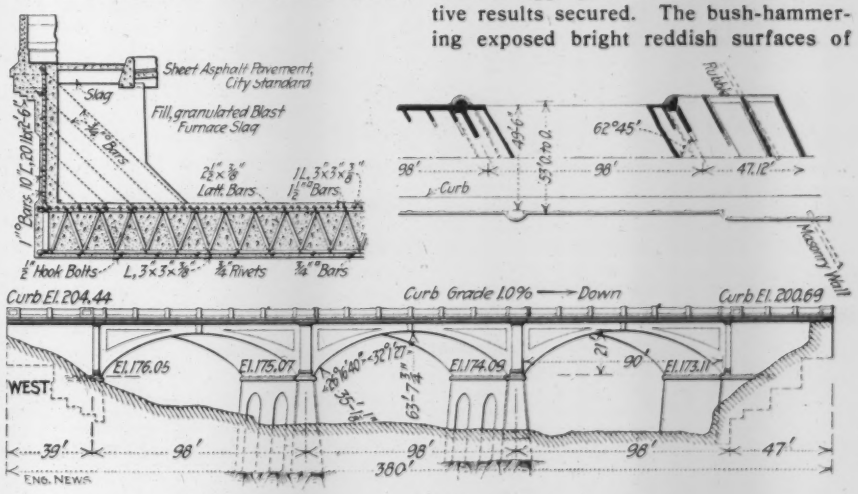


FIG. 16. ATHERTON AVE. BRIDGE OVER PENNSYLVANIA R.R.

the aggregate, contrasting well with the gray cement-mortar parts of the face.

At the same time other sample blocks were made with brick dust in place of sand and gravel. A uniform red surface was obtained, giving an effect much like that of red sandstone.

Brick and terra-cotta débris being available in some quantity in the vicinity of Pittsburgh, it was decided that brick-dust and brickbat concrete could be used advantageously in securing desirable architectural effect in bridges of such location as to demand special regard to the artistic.

This material was first specified for a 60-ft. skew-arch bridge on Hoeveler St., whose construction was begun early in autumn, 1912, by the contractor, the C. M. Neeld Construction Co. The arch ring is of Ligonier stone concrete, showing a gray color; the projecting parts of the coping, the balustrade, etc., are of brickbat concrete; the spandrel walls and other panels are of brick-dust concrete. All showing faces will be rubbed to smooth, even-toned surface. Brick concrete will probably also be used in several other concrete bridges to be built in the near future.

City Commission Plan Charters were voted upon at Los Angeles, Calif., and Duluth, Minn., on Dec. 3. At Los Angeles, the proposed charter was rejected, two to one. At Duluth, the proposition carried, and the new charter will go into effect Jan. 3, 1913, although the present city officials will be continued in office until April 3, 1913.

Diamond-drill Borings are being made at Chicago for two intake tunnels under Lake Michigan, as an addition to the water-supply system. One of these tunnels is on the north side, at Bryn Mawr and the other is at 35th St., to serve the stock-yards district. It is desired to have the tunnels in solid rock, and to avoid the seamy and irregular strata, which have caused so much trouble in the earlier tunnels as described in Engineering News. For this reason the dril holes are made deep enough to indicate that there will be solid rock above and below the tunnel grade.

The drilling extends to a distance of three miles from shore in each case, the maximum depth of water being 38 ft. The deepest hole is 138 ft. deep, or about 100 ft. below the bed of the lake. Many boulders are encountered, and large pockets in the rock; also some ledges of rock about 2 ft. thick, which at first appear to be solid rock, but are found to be underlaid by water-bearing sand. The drilling rig is mounted on a platform, 14x14 ft., supported by 65-ft. piles driven until their tops are about 10 ft. above the water. The tripod is of 30-ft. cedar poles, so as to give ample headway for handling 24-ft. lengths of drill rods. The drill is of the Standard Diamond Drill Co.'s make; it is unusually light, weighing only about 350 lb., so that it can be handled easily and taken to and from the work each day on a scow. The work of the lake and land borings is in charge of F. A. Smith, Assistant Engineer in the Bureau of Engineering, of the city of Chicago.

A Theoretical Formula for the Flow of Liquids through Narrow Rectangular Channels

By Ernst Jonson*

Lamb's "Hydraulics" gives the theoretical formula for flow of liquids through circular tubes. The practical application of this formula is, however, limited because it presupposes absolute uniformity of flow, a condition which is seldom realized. It is only when the velocity is very small that the disturbances in the flow caused by irregularities in the tube are practically negligible. This formula gives, however, the possible maximum flow in a tube, a quantity which will be approached to the extent that disturbing influences are eliminated.

The writer has developed the theoretical formula for the flow of liquids through rectangular channels. This formula presupposes that the width of the channel is only a small fraction of the depth, as is the case with the annular channel between a piston and the inside wall of a cylinder or the expansion joint in a dam. It also presupposes the same conditions as those by which Lamb's formula is governed, namely, that the flow shall be uniform or approximately so. Its practical application is, therefore, limited to very narrow channels or cracks where the velocity of the flow is slight. The limit of applicability of this formula may be determined under any given requirements of accuracy by figuring the theoretical velocity and then determining the corresponding empirical loss of head due to lack of uniformity of flow resulting from irregularities in the channel or other disturbing conditions. The formula is as follows:

$$F = \frac{2\,pr^3}{3\,lu}$$

in which

F = Flow per second per unit of depth of channel;

p = Difference in pressure at the two openings of the channel;

r = Hydraulic radius or half width of channel;

l = Length of channel in direction of flow;

u = Coefficient of viscosity, which in the case of water may be taken as

$$u = \frac{0.000013}{18 + T}$$

when using inches and pounds as units, T being the temperature of the water in degrees Fahrenheit.

This formula is now published because it has been experimentally verified. The following confirming experiments were made by the Board of Water Supply, City of New York. A bronze piston 6 in. in diameter and 8 in. long was placed in a bronze cylinder with a clearance of about 0.002 in. Filtered water at a temperature of 74° F. was then forced through this annular channel under pressures varying from 3.5 to 120 lb. per sq.in., and the flow measured. The following table gives the calculated as well as the measured flow of various pressures.

Pressure lb. per sq.in.	Flow in cu.in. per sec. Calculated	Measured
20	0.21	0.14
40	0.42	0.35
60	0.62	0.61
80	0.83	0.83
100	1.04	1.04
120	1.25	1.28

The actual micrometer measurement of the diameter of the piston was 5.9971 in., and that of the diameter of the cylinder 6.0007 in., making the width of the annular channel 0.0018 in. Since the micrometers used in making these measurements could be read only to $^1/_{1000}$ part 805 C 534 C W B No cuts Set Nov 15 of an inch, and since the general run of the observations indicate that this measurement is somewhat small, it was assumed to be about $^1/_{10000}$ part of an inch too small, namely, 0.001946 in., and the calculated values are based on this latter figure, which was derived from the flow. In other words, through the formula the flow became a more accurate measure of the width of the channel than the micrometer used.

A second test was made with another piston differing from the first mentioned one in that eight circumferential grooves $^3/_8$ in. wide were cut in its cylindrical surface, thus reducing the remaining thin annular channel to an aggregate length of 5 in. The diameter of the second piston was 5.9967 in., thus making the annular channel 0.002 in. wide, or, more exactly, 0.001956 in. as derived from the flow of water. The following table gives the calculated as well as the measured flow of various pressures; temperature of water, 74° F.:

Pressure lb. per sq.in.	Flow in cu.in. per sec. Calculated	Measured
20	0.33	0.27
40	0.66	0.57
60	0.98	0.91
80	1.31	1.31
100	1.64	1.64
120	1.97	1.94
Continued after 2 days' intermission		
20	0.33	0.31
40	0.66	0.62
2.75	0.07	0.06
3.0	0.07	0.06
Continued after 1 day's itnermission		
60	0.98	0.90

Previous to this series of tests, another series had been made with each piston differing from the above series in that unfiltered city water was used. The results of these tests are given in the table, "Tests with Plain Piston."

It will be seen in comparing the column of calculated flows with that of the measured flow, that the latter are considerably less than the former, and that when higher pressures are put on the

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TESTS WITH PLAIN PISTON

Temperature of water 65° F.

Pressure lb. per sq.in.	Flow in cu.in. per sec. Calculated	Flow in cu.in. per sec. Measured	Measured / Calculated
30	0.28	0.14	0.50
40	0.38	0.17	0.44
60	0.56	0.28	0.50
80	0.75	0.38	0.51
100	0.94	0.50	0.53
120	1.13	0.61	0.54
140	1.32	0.85	0.64
140	1.32	0.80	0.60
120	1.13	0.64	0.56
100	0.94	0.48	0.51
80	0.75	0.35	0.47
60	0.56	0.25	0.45
Continued after 2 days' intermission			
40	0.38	0.14	0.36
40	0.38	0.11	0.28
40	0.38	0.11	0.28
80	0.75	0.20	0.26
140	1.32	0.73	0.55
200	1.88	1.17	0.62
40	0.38	0.18	0.47
180	1.69	1.02	0.60
60	0.56	0.29	0.51
160	1.50	0.90	0.66
80	0.75	0.38	0.50
140	1.32	0.66	0.50
120	1.13	0.49	0.42
100	0.94	0.35	0.37

measured flow approaches the calculated. A probable explanation of the difference between calculated and measured flow is found in the fact that unfiltered water was used in the test, and hence, the flow being retarded by dirt caught in the channel. It is natural to assume that the number of obstructions would decrease as the velocity of the flow increases. This conclusion is confirmed by the observation that whenever the pressure is increased the resulting scouring action of the water increases the capacity of the channel, bringing it nearer to the theoretical capacity; and prolonged testing under low pressures such as is seen in the beginning of the second group of tests tends to bring the actual capacity farther below the theoretical.

TESTS WITH GROOVED PISTON

Temperature of water 64° F.

Pressure lb. per sq.in.	Flow in cu.in. per sec. Calculated	Flow in cu.in. per sec. Measured	Measured / Calculated
27	0.41	0.22	0.54
40	0.60	0.29	0.49
60	0.90	0.40	0.44
80	1.20	0.47	0.39
100	1.50	0.57	0.38
120	1.80	0.67	0.37
140	2.10	0.67	0.32
140	2.10	0.59	0.28
140	2.10	0.54	0.27
120	1.80	0.37	0.20
Apparatus cleaned.			
140	2.10	1.37	0.65
120	1.80	0.90	0.43
100	1.50	0.61	0.40
80	1.20	0.38	0.32
60	0.90	0.23	0.25
40	0.60	0.10	0.17

By comparing the theoretical flows with those actually observed, it is found that these latter fall still more below the theoretical ones than was the case with the plain piston. This is evidently due to the fact that in the case of the grooved piston there were nine entrance points where particles of dirt are liable to be caugnt instead of one such point. The fourth column in the above table gives the measured flow in percent. of the theoretical flow and shows how the percentage decreases as the test goes on and is again brought up by the cleaning of the apparatus at the beginning of the second period. The shortage in the first measurement of each period is evidently due to the fact that the water had been flowing some minutes before the measurement was taken. The loss of head due to the entrance of the liquid into the annular channel was figured, but found to amount to only a few thousand parts of a pound, and it was, therefore, regarded as utterly negligible.

These tests with unfiltered water explain why the flow past a piston may be reduced by circumferential grooves, although such grooves would increase the flow of clean water. The dirt accumulating in such grooves acts as piston rings.

The development of the formula is as follows:

The argument is similar to that used in developing the formula for uniform flow through circular tubes given in Lamb's "Hydraulics."

v = Velocity of flow per second.

The resistance on a surface parallel to the center plane of the channel and located at distance x from that plane is

$$f = -l\frac{dv}{dx}u$$

The pressure on a central sheet of water of the thickness $2x$ is

$$P = 2px.$$

This P must be equal to the reaction $2f$, hence

$$-l\frac{dv}{dx}u = px$$

$$-dv = \frac{p}{lu} \times dx$$

hence,

$$v = C - \frac{p}{lu}\frac{x^2}{2}$$

When $x = r$ the velocity must be zero, hence

$$C = \frac{p}{lu}\frac{r^2}{2}$$

$$v = \frac{p}{2lu}(r^2 - x^2)$$

The average velocity is, therefore,

$$v = \frac{p}{2lur}\int_0^r r^2 dx - x^2 dx$$

$$v = P\frac{r^2}{3lu}$$

And the consequent flow per unit of depth of channel per second is

$$F = \frac{2}{3}\frac{pr^3}{lu}$$

The Employees School of the American Bridge Co., at the Gary (Ill.) plant, opened its second season in September. The instruction here is planned to meet the requirements of any man in the shop or office—whether they have had little schooling or have taken high-school and university work.

Three courses, in general, are offered; textbooks and instruments have to be provided by the students, but all other supplies are furnished by the company. Course I takes up Arithmetic, Reading Blue Prints, Business Correspondence and Reports and Elementary Drawing. It is adapted to shop men who have not completed the eighth public-school grade. Classes meet in one of the main-office drawing rooms on Monday and Wednesday evenings from 6:30 to 8:30. Course II is for men more advanced than in Course I; it covers Algebra, Geometry, Plain Trigometry, Strength of Materials, Business Correspondence and Reports, and Mechanics. Classes meet for two hours on Tuesday and Thursday evenings. Course III is for members of the drafting forces, shop foremen, and shopmen who have completed the work of Course II. It takes up Estimating, Structural Design and Computation of Stresses, and Public Speaking. Classes are held from 6:30 to 8:30 p.m. on Mondays and Thursdays.

Good attendance is compulsory for all who enroll; two unexcused consecutive absences are sufficient reason for dropping a student from the courses. Monthly reports are made to foremen or department heads, giving for the respective department each student's grade and attendance.

Ultra-violet Ray Water Disinfection for Railway Trains—Considerable attention is being given in France to practical applications of the sterilizing action of the ultra-violet rays, and a sphere in which it is thought they may render excellent service is the purification of the drinking water supplied to travelers in railway stations. It is a difficult matter to guarantee that the water utilized for this purpose is free from infectious germs. The French railway companies generally use some type of porcelain filter for purifying the water, but the output of these is very small and the cost of upkeep is great owing to the constant attention they require. To obviate these drawbacks the Nord Ry. Co. has decided to adopt sterilization by ultra-violet rays, and installations are being prepared for their Paris terminus and other large stations which possess the requisite supply of electricity.

The rays are to be produced by means of quartz mercury vapor lamps of the Westinghouse Cooper-Hewitt type. The apparatus consists of a quartz lamp tube, from which the rays are emitted, surrounded by a cylinder containing a translucent quartz pane, through which they pass directly into the water. The lamp and outer jacket are fixed centrally in the axis of a small cylindrical water reservoir. By means of a special arrangement of vanes the water is caused to pass four times in front of the quartz pane before reaching the outlet pipe. An automatic valve stops the flow of the water when the lamp is not working, and the water then runs to waste. As soon as the lamp lights up the water again flows through the reservoir. This arrangement insures that all the water supplied by the apparatus has been sterilized.

The apparatus, which is made by the Société Francaise pour les Applications des Rayons Ultra-violets, takes up little space, and can supply 600 lit. of sterilized water per hour with an expenditure of 385 watt-hours of electrical energy. Its actual working cost is therefore insignificant. The cost of the installation, not including the water pipes or electric wiring, is about 1000 francs.

Bacteriological analyses show that the water when it leaves the apparatus is effectively sterilized, and it is not in any way changed to the taste.—"The Times Engineering Supplement," London.

Underpinning the Cross Building, Fifth Avenue, New York

On the site of the old Café Martin, on the southwest corner of 26th St. and 5th Ave., New York City, there is now being built a 20-story office building. The foundations are to be built in open cut, running down to the rock which lies on a varying level from 10 to 50 ft. below the curb elevation and as low as 40 ft. below the concrete footing of the adjoining Cross Building. This building is, as shown in Fig. 1, a narrow wall-bearing structure 13 stories high above the street level. At its front and back, the side walls for a distance of a quarter of the depth of the building are carried on the property line from the footing to the roof on what was intended to be a party wall, but for the middle half of the building a light shaft has been provided and the walls on the property line extend only through the fourth story of the building, the main section of the upper nine stories being carried on steel columns and beams with facing-brick walls 4 ft. back from the building line. The front and back walls of the building on 5th Ave. and Broadway, respectively, are carried on steel girders spanning concrete footings resting on the regular pile foundation.

The problem in constructing the foundation of the new building was to support this bearing wall through its whole length during the excavation of the cut for the new building, and at the same time to provide a stable permanent footing under this wall at the elevation of the footings of the new building. It was decided not to utilize the party-wall feature of this side wall, but merely to carry it down to the new footing and to build the side wall of the new building immediately adjacent to the old wall. As the new building is of skeleton steel construction, the wall to be built was merely a closure wall taking up very little space.

Fig. 2 represents in diagrammatic form the successive steps taken in the underpinning operation on the main section of the wall. The first step was to cut through the old brick wall, which was 56 in. thick, an opening large enough to allow the entering of the needle beams, made up of four 24-in. I-beams. This wall was hard brick laid up in cement mortar and it required some blasting to make the opening. The needles, which were spaced about 6 ft. apart along the wall, were supported on the inside of the old building on blocks placed on the concrete cellar floor, and on the outside on blocks supported on the earth immediately alongside the wall. Sheathing was then driven outside of the blocks, and an excavation made to solid rock. On this rock a rough concrete footing was placed and 12x12-in. posts erected to carry the outside end of the needle beams, the needle beams then being supported on the inside by the blockings of the concrete pavement and on the outside by the heavy posts to a solid concrete footing. Shims were driven in under the brick wall for support and sheathing driven on the inside of the old building, as shown in the third view in the diagram. Excavation was then made under the brick walls to rock bottom, and the entire old footing removed. A new concrete footing was placed at this rock bottom, which, as has been stated above, was in some places as low as elevation −50 and the new brick wall carried up to the underside of the old wall.

FIG. 1. FOUNDATION WORK ON THE NEW BUILDING, 26TH ST. AND FIFTH AVE., NEW YORK CITY

(Fifth Ave. in foreground to left, 26th St. to right, Broadway in the background.)

Meanwhile the walls were strengthened by very long yellow pine 14x14-in. struts which extended from a point 40 ft. above the curb level to the concrete footings on the bottom of the new excavation, a distance of about 70 ft. These, together with the details of the framing and sheathing after all excavation was made, are shown in Fig. 3. When a section under the wall 15 to 20 ft. long had been excavated in this fashion and the new brick wall carried up to the old one, the load of the latter was transferred to the new foundation through a series of granite wedging blocks placed between the needles. These blocks are 12 in. thick, 3 ft. wide and the thickness of the wall. They are placed upon the new lower wall and wedged tight with steel shims driven hard between the upper and lower block. When they were tight in place, the needles were removed and the wall carried upon the wedges until the opening was filled in the manner common in such underpinning work.

In the middle section of the building, where the light well had been placed, the method of underpinning differed in its details from that used under the heavier walls at the ends. In the first place the outside brick wall, 24 in. thick at the bottom, was pierced and supported in the same manner as the 54-in. main brick wall. The complications resulted from the supporting of the interior wall, which was of brick resting on stringers framed into steel columns, which latter also carried floor beams of steel construction. The method used is shown in Fig. 4. It will be noted from that drawing that the floor beams were supported on 12x12-in. posts resting directly on some 12-in. I-beam stringers which in turn spanned the needle-beams, and that the wall-bearing stringers were supported on posts resting on 14x14-in. timbers, which spanned the aforementioned 12-in. I-beams.

The needle-beams were spaced so as to avoid the wall columns except in one place where the motor room for the elevators in the old building, which had

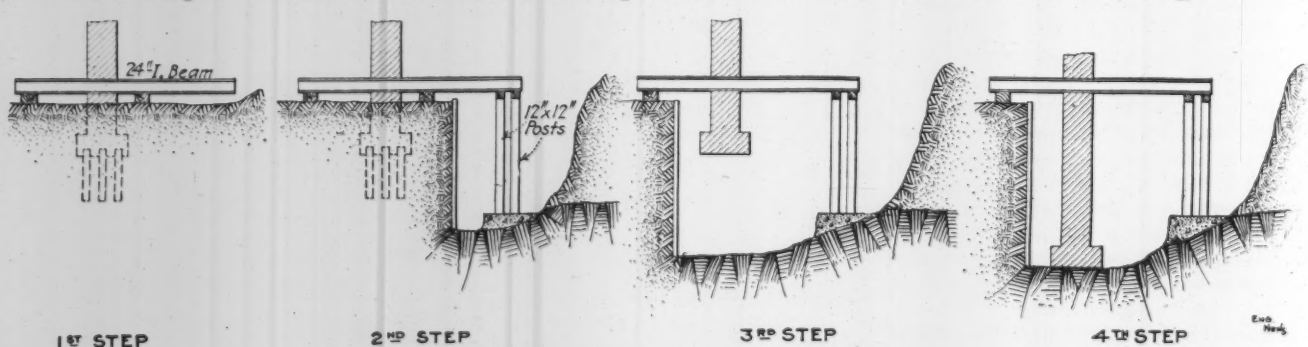


FIG. 2. DIAGRAM SHOWING PROGRESS OF UNDERPINNING

to be kept constantly in operation, interfered with the proper placing of the needles. Here the only place to pierce

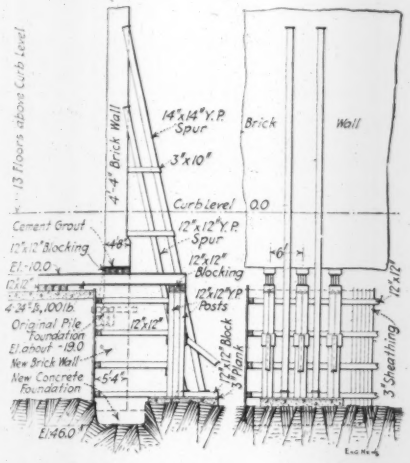


FIG. 3. DETAILS OF UNDERPINNING AT 56-IN. WALL SECTION

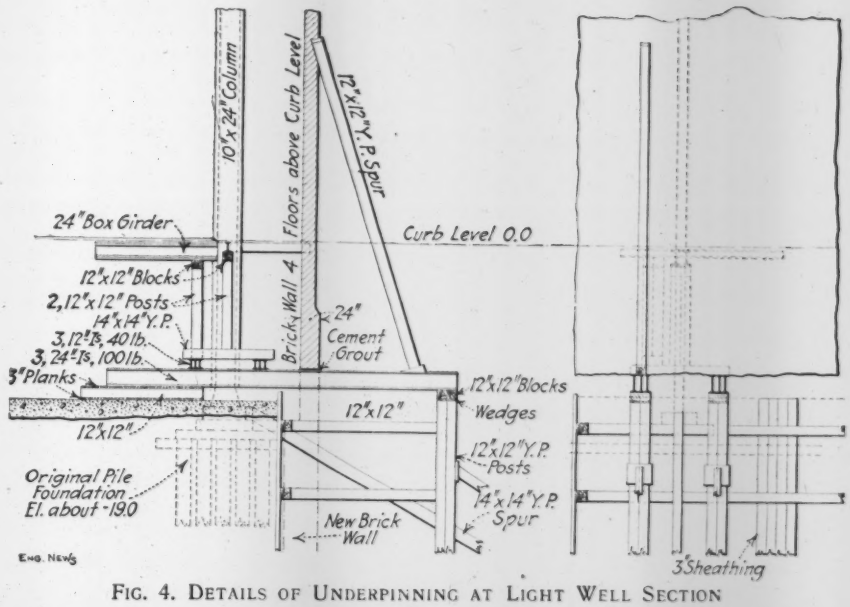


FIG. 4. DETAILS OF UNDERPINNING AT LIGHT WELL SECTION

the wall was through one of the interior columns.

This piercing was a very delicate piece of work, for which several solutions were suggested. The one finally adopted is shown in Fig. 5. The column was first reinforced by riveting to it on each side an angle and plate for some distance above and below the needle-beam elevation. A section 8x26 in. was then cut with an oxyacetylene flame clear through the column and its reinforcement and the 24-in. I-beam needle shoved through the opening and supported on each side as in the other needles on the building.

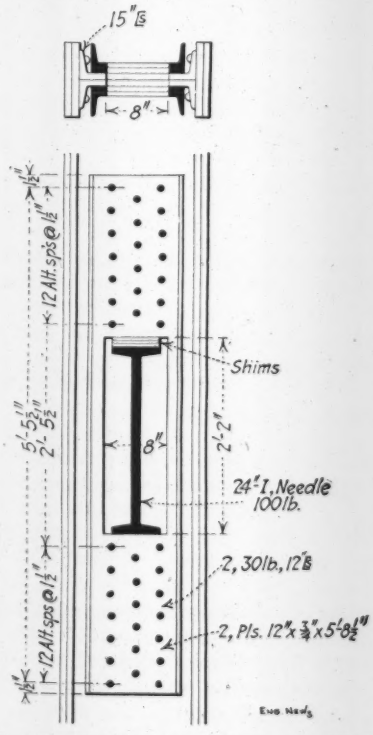


FIG. 5. METHOD OF SUPPORTING WALL COLUMN

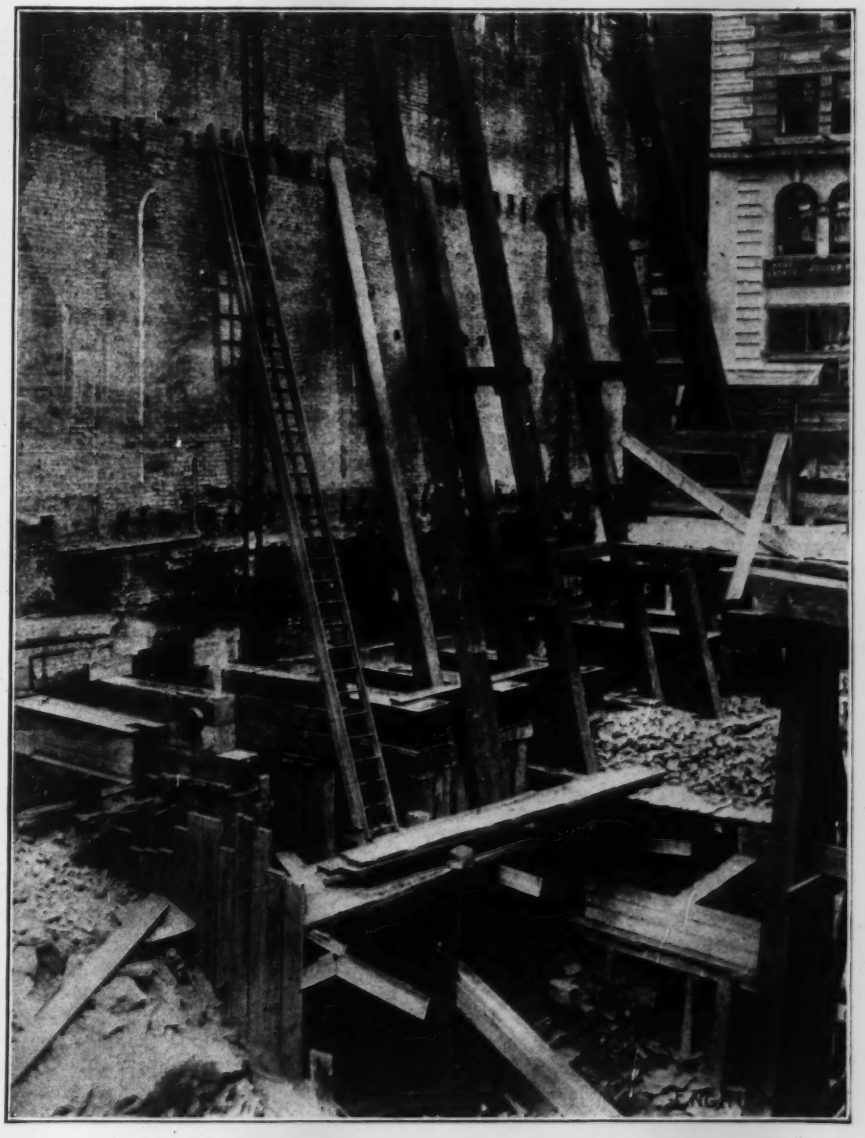


FIG. 6. VIEW OF UNDERPINNING AT BROADWAY END OF BUILDING

The column was supported on the needle by steel shims as shown. As much of the load as could possibly be taken was supported by the adjacent needles placed on either side of the column and shored in the usual manner.

The main difficulties with the work were connected with the necessity for blasting the rock under the walls of the old building and insuring the stability of this building during the process, as well as quieting the fears of the residents of the upper ten floors of the Cross Building, which is used as an apartment house. On the Broadway end of the excavation, that is, in the background in Fig. 1, an outcrop of rock appeared very slightly below the elevation of the old pile footing, so that an excavation of 22 ft. in rock had to be made at this point to reach the required bottom elevation. This rock was blown out in small blasts with no apparent damage to the building nor distress to its occupants. Channel borings on 4-in. centers were made along the face of the rock for the blast. On the 5th Ave. front of the building the rock suitable for foundation dropped down to a depth some 50 ft. below curb level, and required the heaviest type of sheathing and cross-bracing to hold back the street and adjacent building. All this cut was made, however, in open pit.

The cellar of the Cross Building was filled with machinery, piping and sewers, all of which had either to be avoided by the needles or changed in places where a certain position for the needle was imperative. All of this changing work was done without disturbing the occupants of the building, and furthermore no cracks have appeared in the side walls of the building. The wood piles under the old building were found to be in bad shape from hard driving, despite the fact that many were not driven to rock, but had not shown any signs of rotting. The walls of the new building are to be built immediately alongside of those of the old with a multiple layer of waterproofing between in order to exclude from the new cellar any moisture and at the same time to retain the ground-water level of the old building above the elevation of the piles, which are under the other wall only about 30 ft. away.

The work is being done for the Twenty-Sixth Street & Fifth Avenue Realty Co., by Canavan Bros. Co., who have the contract for the entire excavation as well as the underpinning. Thomas F. Shannon is superintendent of the shoring and A. C. Franke and Arthur H. Diamant are engineers for the company and are in direct charge of the underpinning and excavating work.

City Surveying in Brooklyn*

By John B. Stein†

The duties and organization of the Topographical Bureau, Borough of Brooklyn, New York City, are here described, together with an account of the work done by the bureau, past and present; the results of the triangulation of the borough; the changes in the city plan; the methods of surveying and the adjustment of monumented base lines; the various standards of measurement used, and the preparation of plans and profiles for original improvements. The Borough of Brooklyn, formerly the City of Brooklyn, has an area of 81 sq. mi., and represents a consolidation of several distinct towns and villages, each originally mapped and planned without relation to one another.

*A paper read before the Brooklyn Engineers' Club, Oct. 10, 1912.

†Assistant Engineer, Topographical Bureau, Borough of Brooklyn, New York.

The Topographical Bureau of the Borough of Brooklyn is a comparatively new department. In 1902, on the dissolution of the general Topographical Bureau of Greater New York, a separate bureau was established for each borough, and the Topographical Bureau of Brooklyn became a division of the Bureau of Highways, and in 1906 was created an independent bureau of the Borough President's office.

The work of this bureau is the completion of the city plan, by laying out and establishing grades for new streets, parks, and public places; the readjustment of the city plan by making changes in the lines and grades of streets, where conditions demand; the preparation of maps for use in the condemnation proceedings for acquiring title to streets, parks and public places, as laid out upon the city plan; the preparation of plans and profiles for the original improvement of streets and public places; the monumenting and re-monumenting of the street system; the establishment of a system of level benches, and the compiling, recording, and indexing of acts of past and present official bodies affecting the borough, for office and public use.

Organization

The organization for carrying out the work as outlined above is based upon the division plan with an expert in charge of each division as follows: A surveying division, a monumenting division, a computing division, a grade divison, an orginal improvement division, a drafting division, a record division, and a clerical division. Fig. 1 shows a chart of the duties of the various divisions.

Surveys—The field or surveying division is divided into four or more field parties, each party consisting of one assistant engineer, one transitman, one rodman or chainman, and one or more laborers. All surveying, cross-sectioning, leveling and the setting of monuments is done by this division.

Monuments — The monumenting division is entrusted with the keeping up of the original records of the locations and conditions of all official street and private monuments, or any change therein, and the recording of new monuments set. Its duties include the care and custody of all monument and traverse sheets, benchmark charts, field books and the indexing of same, and the charting of precise chaining and angles measured. The providing of the field parties of this or other city departments and private surveyors with information to be used for location work and lot surveying is a duty of this division.

Computation—The computing division makes all computations for rule, damage and lay-out maps (described later) such as locating lots, the dimensions and areas of the same, and the calculating of block dimensions and traverses of filed maps. The computations are made by two computers independently of one another, in separate computation books, and all results are checked in this manner, both as to figures and methods.

Grades—The grade division has the work of making changes in the official grades, which is one of the most intricate city surveying problems.

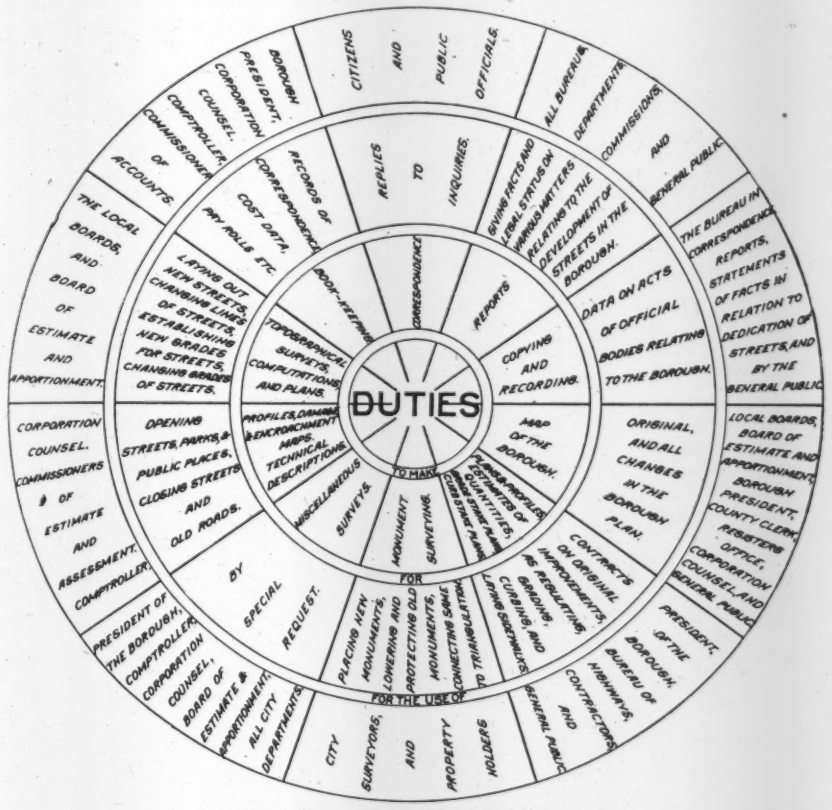
Improvements—The original improvement division has the preparation of all plans and profiles for the regulating, grading and paving contracts let by the Bureau of Highways. This division has charge of making the necessary computations of quantities, the grade and curb staking, the preparation of the monthly and final estimates for grading, curbing, sidewalks and the paving of streets.

Drafting—The drafting division makes all the drawings, maps, plans and profiles; all working maps, profiles and surveys are plotted by this division from data worked up, or to be worked up by the other divisions. When these are checked by the proper division, finished maps or profiles are made ready for the approval of the Chief Engineer, who forwards them to their proper destination.

Records—The record division is in charge of all records, the filing and indexing of same, and the custody of all maps to be kept for reference.

CLERICAL—The clerical division is responsible for bookkeeping, correspondence, cost data, payrolls and supplies.

TIME KEEPING—The system of keeping the time of employees and the progress of work is done in the following manner: Each employee of the Topographical Bureau is provided with a daily time card, on which he records the time he is present at the office, the particular piece of work he is employed on each day, and at the end of each week this time card is turned in to the head of the division and attested to by him, who, from these time cards, makes out his weekly progress report to the Chief Engineer. This report shows all the work under way, the percentage of work done during the past week, and of the previous week, thus enabling the Chief Engineer to follow the progress of all the work and to detect inefficiency in any division.


DUTIES
TO MAKE
FOR
FOR THE USE OF
CITIZENS AND PUBLIC OFFICIALS
REPLIES TO INQUIRIES
CORRESPONDENCE
REPORTS
COPYING AND RECORDING
MAP OF THE BOROUGH
ORIGINAL, AND ALL CHANGES IN THE BOROUGH PLAN
CONTRACTS ON ORIGINAL IMPROVEMENTS AS REGULATING, GRADING, CURBING, AND LAYING SIDEWALKS
GENERAL PUBLIC AND CONTRACTORS
PROPERTY HOLDERS
CITY SURVEYORS
ALL CITY DEPARTMENTS
BOARD OF ESTIMATE & APPORTIONMENT
COUNSEL
COMPTROLLER
PRESIDENT OF THE BOROUGH
BY SPECIAL REQUEST
OPENING STREETS, PARKS, & PUBLIC PLACES, CLOSING STREETS AND OLD ROADS
COMMISSIONERS OF ESTIMATE AND ASSESSMENT
CORPORATION COUNSEL
THE LOCAL BOARDS, AND BOARD OF ESTIMATE AND APPORTIONMENT
LAYING OUT NEW STREETS, CHANGING LINES OF STREETS, ESTABLISHING NEW GRADES FOR STREETS, CHANGING GRADES OF STREETS
COMMISSIONERS OF ACCOUNTS
COMPTROLLER
CORPORATION COUNSEL
BOROUGH PRESIDENT
RECORDS OF CORRESPONDENCE, COST DATA, PAY ROLLS ETC
BOOK-KEEPING
TOPOGRAPHICAL SURVEYS, COMPUTATIONS AND PLANS
PROFILES, DRAWINGS, ENCROACHMENT MAPS, TECHNICAL DESCRIPTIONS
MISCELLANEOUS SURVEYS
MONUMENT SURVEYING
PLACING KEY MONUMENTS, LOWERING AND PROTECTING OLD MONUMENTS, CONNECTING SURVEYS
GIVING FACTS AND LEGAL STATUS ON VARIOUS MATTERS RELATING TO THE DEVELOPMENT OF STREETS IN THE BOROUGH
DATA ON ACTS OF OFFICIAL BODIES RELATING TO THE BOROUGH
PLANNING PROFILES, QUANTITIES, AND CROSS SECTIONS
LAYING SUBSTRUCTURE
THE BUREAU IN CORRESPONDENCE, REPORTS, STATEMENTS OF FACTS IN RELATION TO DEDICATION OF STREETS, AND BY THE GENERAL PUBLIC
ALL BUREAUS, DEPARTMENTS AND COMMISSIONS
GENERAL PUBLIC
LOCAL BOARDS, BOARD OF ESTIMATE AND APPORTIONMENT
BOROUGH PRESIDENT, COUNTY CLERK, REGISTERS OFFICE, CORPORATION COUNSEL, AND GENERAL PUBLIC
PRESIDENT OF THE BOROUGH, BUREAU OF HIGHWAYS, CONTRACTORS AND GENERAL PUBLIC
CITY SURVEYORS AND PROPERTY HOLDERS

COST OF BUREAU PROVIDED FOR BY

FUND FOR STREET AND PARK OPENINGS	32000.00	FOR SALARIES & WAGES	97000.00
FUND FOR STREET IMPROVEMENT	12000.00	FOR SUPPLIES	3000.00
CORPORATE STOCK	56000.00		
TOTAL COST	100000.00		100000.00

FIG. 1. DIAGRAM SHOWING ORGANIZATION SCHEME

IMPORTANCE OF WORK DONE BY THE BUREAU

When it is considered that no improvement of any kind can legally be made in any street of the borough until the title of that street is vested in the City of New York, it is evident that the initiating and progress of all public improvements constructed and maintained by all the other departments of the borough government depend upon the progress of the work of the Topographical Bureau. From a legal standpoint the work of this bureau is to the work of all other departments what the foundation of a building is to its superstructure.

CITY PLAN—The Topographical Bureau through its system of searching, has collected a large mass of data and references relating to the general history of the streets of Brooklyn from the days of the village of Brooklyn, with an area of a little less than 1 sq.mi., to the present Borough of Brooklyn, with an area of about 81 sq.mi. These records show the gradual growth of the street system, and are contained in a series of 95 maps, embodying the original plan and the changes that have taken place up to the present time.

Another series of charts contain a history of the grades of all streets and the changes therein up to date. On a large wall map of the borough the legal status of streets is shown, and the various methods of acquiring title to them, in a color scheme which can be easily recognized. A card index is kept in connection with this chart, showing the exact date and method of acquisition of each street and other public property.

MAKING OLD RECORDS AVAILABLE—The files of the Topographical Bureau now contain 10,000 maps of all kinds, many of which are used and consulted by the public at large almost daily. Some of the records unearthed by the bureau's system of searching had virtually been lost, having been laid away unindexed in almost inaccessible places in the various public offices. From them has been compiled the history of each individual street of the borough. All facts relating to a street are contained in a folder, so that a complete history can be ascertained at short notice. These files also contain references to old roads, railroads, bulkhead and pierhead lines, government reservations, city property acquired for various purposes, other than for streets, and the history of the nine towns and villages which now comprise the borough.

The value of this work has been shown by the constantly increasing number of requests made for information, as the existence of these records becomes better known, not only by the other city departments, but by the public in general, who may now obtain information where formerly the aid of an expert searcher or lawyer was necessary.

FIELD WORK—All the surveying done by the bureau is the result of petitions filed for public improvements; some for changes in plan and grade, others for surface improvements, and requests from almost every city department for information relating to the city plan. Every kind of construction must be done according to the legal lines of the streets.

The advisability of proposed improvements is investigated, reported upon, and forwarded to the Board of Estimate and Apportionment, for authorization. If adopted by the board the resolution is returned to the Topographical Bureau for the preparation of maps for filing, for plans and profiles and for estimate of quantities, for original improvements of streets, and of condemnation maps for street openings.

ADJUSTMENT OF LEVELS TO A COMMON DATUM

In making a study of the grade charts adopted at different times by the various municipalities comprising the Borough of Brooklyn, it was found that there were 10 different charts based upon 6 different datums. Studying the datum situation, we found that there were 13 different datums used by 15 different departments or commissions. These combined make a set of 30 different corrections which must be considered by this bureau in order to refer maps made by different departments or commissions to a common datum plane.*

Upon locating and investigating physi-

*For a description of some of the various datums used in New York City see editorial in Eng. News, May 17, 1906, p. 543.

cal improvements in certain sections, it is found that the improvements were put in and buildings built upon the original surface, irrespective of any legal grade whatever. In many instances improvements were placed upon old legal grades and these grades were found to be not sufficient for drainage purposes, and it became necessary to establish new grades that meet the requirements for proper drainage and sewer construction.

Estimate and Apportionment, for adoption, all grade maps based upon a common datum plane. These maps comprise territories of such size that they can be surveyed and mapped without great delay, thereby adopting in sections a new grade chart for the entire borough.

Foundations of the City Plan

The official map of Brooklyn, as now constituted (Fig. 2), consists of six different maps made by the various authorities, and dating back to 1819. These maps were made independently of one another and at a time when the most far-sighted were unable to discern the future in store for Brooklyn, consequently the more comprehensive methods used by the Boroughs of The Bronx, Queens and Richmond, in making their final maps, were not and could not have been adopted when the maps of this borough were prepared.

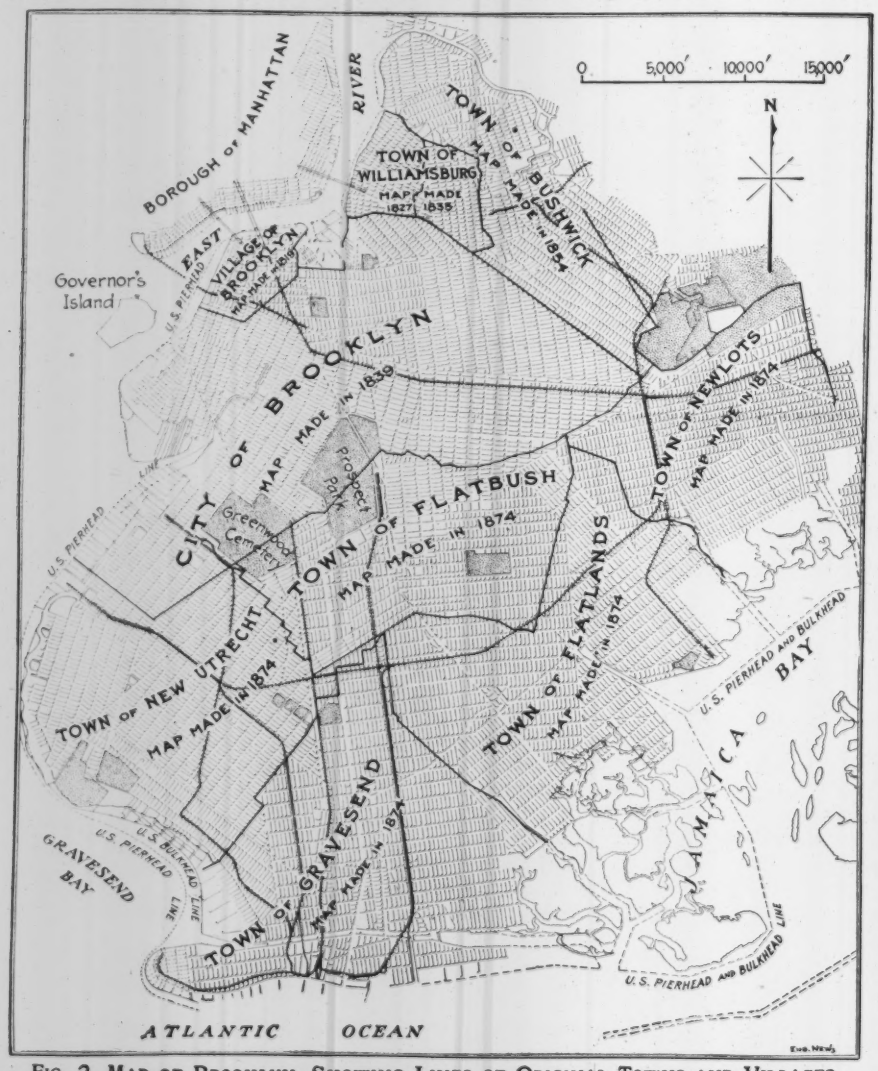


FIG. 2. MAP OF BROOKLYN, SHOWING LINES OF ORIGINAL TOWNS AND VILLAGES AND COMPLICATED STREET SYSTEM

To do little or no damage to buildings as constructed, and to retain as much of the existing improvements as is practicable, to save the property owners the expense caused by making any changes in the construction of the fronts of their buildings or putting down new sidewalks or curbs constitute problems which it can be readily seen requires great care and study.

In order to secure for this borough a uniform system of grades it is necessary to remodel our entire present grade chart. To accomplish this result the Topographical Bureau has adopted a policy of forwarding to the Board of

The great disadvantage of having so many maps is not readily appreciated by laymen, but will be realized to some extent when it is known that since the adoption of these maps, up to the present time, over 1500 changes in the street lines have been made. In some parts of the borough entire sections have been remodeled. In the outlying sections the necessity of preserving the lines of the old roads has been fully realized in the work of widening streets and readjusting the adjacent street system; these outlying roads were practically disregarded by the old official maps. Then again, great manufacturing enterprises and the improvement of waterways have caused many changes in the street system abutting shore fronts.

MAIN TRIANGULATION—Section 439 of the Greater New York Charter, as amended, provided for and made it the duty of the president of each borough to prepare a map, based upon accurate triangulation of that part of the city embraced within his borough. An amendment of this section of the Charter, passed May 8, 1903, made it possible for the city to coöperate with the U. S. Coast & Geodetic Survey in completing the triangulation of the city.

The primary and secondary triangulations conducted under the direction of A. T. Moseman, Assistant of the U. S. Coast & Geodetic Survey, have been completed, and the Topographical Bureaus were furnished coördinates of triangulation stations. For the Borough of Brooklyn the origin is the Prospect Park water tower.* From these coördinates the original street system, as laid out on the six different city maps, have been accurately connected and plotted, with all the subsequent map changes up to date, and the result has been a new map of the borough, plotted absolutely to scale and correctly oriented. The original map was made to a scale of 600 ft. = 1 in., and plotted on mounted paper in two sections, 6x11 ft. each. tracings were then made of this map and lithographic reproductions, in three different scales were printed for general use. This is the first accurate map of the entire borough ever made.

There is under preparation a map of the entire borough, in sheets 25x35 in., on a scale of 200 ft. = 1 in. On these sheets will be shown official streets and parks, block dimensions, angles, monument lines and monuments fixing the lines of the streets, grades and benchmarks. The outlying sections, comprising the five towns, have been taken up first, as most of the work and improvements are going on in these sections.

Resurveying Old Monument Lines

Practically all the territory has been resurveyed, that is to say, all monument base lines have been run out and all existing monuments located and adjusted.

*A description of this triangulation work will be found in Eng. News, May 21, 1903, p. 456; Borough of The Bronx, Dec. 24, 1908, p. 715.

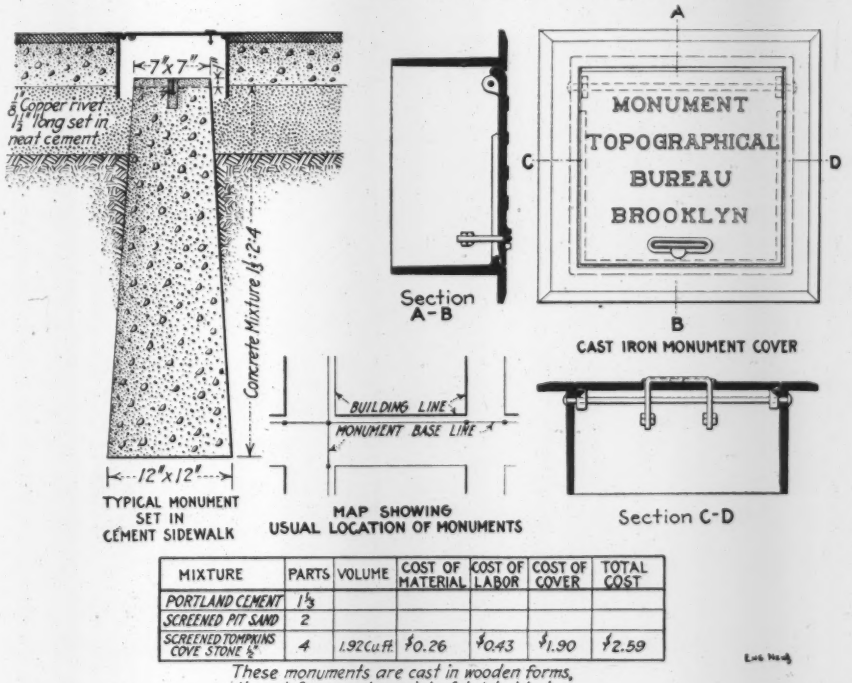
All block dimensions on the original Town Survey Commissioners' map have been retained and most of the missing ones supplied. Existing monuments have been preserved and new ones put in where old ones were missing or none were set.

In laying out the base squares of the commissioners' map of 1874 it was the intention of the engineers to lay them out rectangular; in some cases they succeeded remarkably well, but in many cases they failed. These monumented squares, as shown on the commissioners' map, without any angles, are supposed to be rectangular, but actually are not, and as the monuments set, if not disturbed since the completion of the town survey of 1874, actually and legally fix these lines, the angles have to be taken as they measure.

ADJUSTMENT OF LINES—These squares are relocated or traversed in the following manner: Over each base line taking the extreme two monuments as fixtures, This is done with every line of the figure traversed, and the angles are then measured and used. The lines thus obtained are accurately chained and the distance between the extreme and intermediate monuments noted and compared with the block dimensions as given on the official map, and the shortages or surpluses as they occur are determined and applied to that particular square.

In making the above adjustments the monuments are left as found, that is, they are not moved from their original position. But their relation to the adjusted lines are noted or designated by a copper plug put in the head of the stone, leaving the original intersection cuts, or any other marks upon the stone undisturbed.

NEW MONUMENTS—In sections where the base lines were monumented but the intermediate stones were never set or are missing, this same method is pursued and after adjustments are made and shortages or surpluses determined, these



MIXTURE	PARTS	VOLUME	COST OF MATERIAL	COST OF LABOR	COST OF COVER	TOTAL COST
PORTLAND CEMENT	1⅓					
SCREENED PIT SAND	2					
SCREENED TOMPKINS COVE STONE ½	4	1.92 Cu. ft.	$0.26	$0.43	$1.90	$2.59

These monuments are cast in wooden forms, the reinforcement consist of twisted bale wire or any other heavy wire available.

FIG. 3. TYPICAL STREET MONUMENT AND METHOD OF SETTING

provided there are no direct evidences that either one of these two has ever been disturbed, a transit line is run and all the intermediate monuments located with reference to that line. This line is then plotted in profile, the horizontal scale being 200 ft. = 1 in., and the vertical scale, representing the offset from the transit line to the intermediate stones 1 ft. = 1 in. Then by stretching a fine silk thread over these monuments as plotted a line intersecting most of these stones can easily be obtained graphically; and in this manner the actual base line can be reproduced as nearly as it was supposed to have been laid out originally. stones (Fig. 3) are set by the bureau, with the shortage or surplus proportionately distributed over the entire base line.

If building operations or grading necessitates the resetting of monuments they are raised or lowered, and their new elevation noted, but they are kept in their original positions horizontally. If it becomes necessary to move them from their original positions they are set on 10-ft. offset lines relative to their original positions.

STREET INTERSECTION PROBLEM — A problem presents itself when changes in the direction of street system occur, as for instance where two rectangular systems intersect obliquely, which frequently happens where the old town lines come together. The commissioners' map in most of these cases very carefully omitted the block dimensions on the connecting street, but laid out and monumented the intersecting lines, although in some cases even these were omitted.

The method adopted by the Topographical Bureau to lay out and fix block dimensions for these skew systems is as follows: Assuming that each rectangular system is adjusted as above, the outlines of the skew system are produced to intersections with the rectangular system, the distances from the last monuments of the rectangular system to the intersecting lines which complete the block triangles are then measured, and this measurement is reduced to the standard of the same side of the larger triangle as determined by the monuments. With these adjusted distances, plus the remaining block distances to the vertex of the triangle, as two sides of a right-angle triangle the remaining two angles and the third side of the complete triangle are computed. In most cases it is found that the computed distance of the above side does not agree with the distance as obtained from the rectangular system adjacent, that is to say, that the rate of shortage or surplus is not the same on both sides of the limiting street between the two rectangular systems. It then becomes necessary to hold one rate for the computation of the block dimensions on the skew side of the system and another rate for the rectangular. This, of course, would make the angle slightly different, according to the system with which it is considered, but it is the better practice to hold distances than angles.

STANDARDS OF MEASURE

At the beginning of this century, Kings County was an agricultural community, consisting of farms, meadow and woodlands, with a few small villages. The first farms to be cut up into lots were situated approximately between Fulton, Tillary and Navy Streets, the Navy Yard and East River. This tract was laid out in 1788 into blocks of several lengths and widths; by comparison of several of these distances the standard was found to be about 4 in. longer in 100 ft. then U. S. standard. In 1805 and 1806 an estate in the vicinity of Fulton Ferry was cut up into building lots, and in laying out the blocks 200 ft. square an extra allowance was given "for good measure" of about 1 ft. in 100 ft.

The old village of Brooklyn was incorporated in 1816, and in 1819 the trustees' map was made showing the streets and roads then in existence. Prior to 1834 when the City of Brooklyn was first incorporated most of the land included within the old village of Brooklyn and several of the then outlying districts, had been laid out into blocks and lots accord-

ing to various standards, most of them by the "New York standard," which was determined by Hassler, then Superintendent of the U. S. Coast & Geodetic Survey, to have a ratio of 36 to 36.01545 of the Troughton scale, which was afterward adopted as the U. S. standard, and was taken by some of the old surveyors to bear the relation of 36 to 36.0288 of the U. S. standard, or 8 in. long in 1000 ft.

The area between Third, Fifth and Atlantic Avenues and Douglass St. was laid out by this standard, which in a distance of 2000 ft., gives a surplus at Douglass St. of 1 ft. 6 in., which was deducted from the next block (making the lots on the north side of Douglass St. 98 ft. 6 in. instead of 100 ft.) in order to make the distance from Atlantic Ave. to Douglass St. correct according to U. S. standard.

In 1835 the legislature appointed a commission to lay out the city of Brooklyn in a regular system of streets. They devised a plan for the city, adopting such streets as had been previously laid out as far as possible. Under their direction, John Stoddard, from 1835 to 1838, laid out and monumented the entire city, excepting such tracts as were previously laid out. He secured from the U. S. government a yard scale from which he had 10 five-foot steel bars constructed, with which he standardized his chains and measures. It is considered that the Stoddard standard was U. S. and was found to be so by S. McElroy, who compared the Stoddard bar with the standard procured by him for the Town Survey some years later.

The first tract laid out into lots in the Eastern District was the original village of Williamsburgh, shown on the maps of Loss, Bridges & Poppleton, dated 1802-6-9. These lots and blocks were of irregular shape and the standard used seemed to be about 1 in. longer in 100 ft. than U. S. standard. In 1834 the village of Williamsburgh was laid out into streets by Daniel Ewen, according to a system shown upon his assessment map, adopting the old streets already laid out on the old Williamsburgh tract. He used the New York standard, which has been found to be about 1 in. longer in 100 ft. than the U. S. standard. In 1834 the additional part of Williamsburgh was laid out by the same standard.

The Town of Bushwick was laid out into streets, in accordance with the direction of a commission in 1852; several farms, particularly in Greenpoint, had been laid out into lots and blocks before that time, by Ewen & Betts who used the New York standard. The northerly section of the town was laid out by the Morgan brothers, who used a standard about 1¼ in. longer in 100 ft. than U. S. standard; and the southerly section was laid out by Martin G. Johnson, who used Ewen's Williamsburgh standard, 1 in. longer in 100 ft. than U. S. standard.

The Town of New Lots, or 26th Ward of Brooklyn, was laid out into streets, according to a system originated and developed by Martin G. Johnson. He used a standard, determined by McElroy to be 1⅛ in. longer in 100 ft. than U. S. standard, a line given by Johnson as 5450 ft. having been found to measure 5454.98 ft. by the U. S. standard.

In 1869 an act was passed appointing a commission to lay out the county towns in a system of streets conforming to the streets of Brooklyn as far as practicable. A system of streets and avenues was devised, maps prepared and the territory surveyed by S. McElroy & Sons, who procured a standard from the U. S. government. In the territory where streets had already been laid out, such as parts of the villages of Flatbush and Fort Hamilton, etc., the Town Survey Commission accepted the streets as existing, and some of these present many varieties of standards, for instance: in the Fort Hamilton village the measure was found to be 1⅛ in. longer than U. S., north and south, and 1¼ to 1½ in. longer east and west; in Parkville, 1 in. longer in 100 ft. than U. S., east and west, and 1¼ in. longer north and south.

There seems to be an entire absence of uniformity in standards used to lay out the different sections, now comprising the Borough of Brooklyn, and if the measurement of land were made strictly according to the U. S. standard, great discrepancies would be disclosed, and endless litigation would result. From the use of the various standards and errors of those surveyors who originally laid out the farms into lots and blocks, a confusion has arisen; and it has become the practice of the Topographical Bureau, as well as the private surveyors of Brooklyn, to consider each block as a standard for itself, taking its dimensions, as defined by the official maps, no matter what it may actually measure in U. S. standard, and distribute the surplus or shortage *pro rata* as far as practicable.

FIG. 4. EARLIEST MAP SHOWING BOTH MANHATTAN AND BROOKLYN

CONDEMNATION MAPS

These maps are made for the purpose of acquiring title for the City of New York to the land within the lines of the streets, avenues, public parks and places, as laid out on the map of the City of New York, in order that improvements, such as sewers, curbs and sidewalks and pavements may be legally installed and also that no further encroachments may be made on these streets.

Before the maps are prepared the Local Board of Improvements is petitioned for the opening of a street. This board passes a resolution to that effect, and presents it to the Board of Estimate and Apportionment, which authorizes the opening and fixes the area of assessment, the Topographical Bureau is called upon to prepare the necessary maps.

The maps required for opening proceedings are: five copies of a rule map, four copies of a draft damage map, two copies of a damage profile (not always required and only furnished upon request), two copies of a final damage map, one draft benefit map and three copies of a final benefit map. The cost of preparing maps for opening proceedings is 12c. per lin. ft. of street.

RULE MAP—The rule map is a small scale map, made on tracing cloth, showing the entire area to be acquired, in outline. The overall dimensions and angles sufficient for a technical description, the intersecting and abutting streets, section and block numbers, and buildings if located within the affected area. The tracing-cloth copy is kept and placed on file in the office and the requested number of prints are forwarded. This map accompanies the application to the Supreme Court for the appointment of three condemnation commissioners.

DRAFT DAMAGE MAP—The draft damage map is for the use of the above commissioners to determine the amount of awards to be made for the land or buildings condemned. This map is made in the following manner: A working map on detail paper is plotted to a scale of 30 or 50 ft. = 1 in., showing the official lines of the street to be opened, the intersecting and abutting streets and widths, block and section number of the Kings County land map, the lots facing the street as shown upon the tax map, their numbers, dimensions and owners' names, farm and old road lines, railroad rights-of-way, buildings, fences and other physical improvements, as determined by survey, within or adjacent to the area affected.

The lines of the lots are extended to the center line of the street as laid out, or to a line determined by deeds or filed property maps, and these lines form the parcels to be condemned. The dimensions and areas of these parcels are computed and the legal and physical notes for the same determined to guide the commissioners in making the awards. The computations and adjustment of these lines are often complicated, as the tax-map data in many cases disagrees with the data given on filed property maps, by deeds or by actual possession, and it is often necessary to go into the determination of lot lines quite extensively. A tracing is made of the working map, and four prints of the same forwarded; the tracing being kept and filed as the original in the office.

DAMAGE PROFILE—A damage profile accompanies the above map (when requested), and it is used by the commissioners to determine any damage to the abutting property by reason of the intended regulating and grading of the street. This map shows all the legal grades ever established, a center-line profile of the actual surface and elevations of entrances of all the buildings facing the street.

FINAL DAMAGE MAP—After the commissioners have finished their work final damage maps are prepared to accompany their report. These maps are made in sheets of a standard size 11x18 in. and are practically a copy of the draft map, with the exception that only the property and buildings taken are shown and such changes as are decided upon by the commissioners.

BENEFIT MAP—The benefit map is made and used for the purpose of levying assessments for the opening proceedings and covers the area determined by the Board of Estimate and Apportionment. The original or draft map is kept and filed for record in the office. The final maps are made of the same size, 11x18 in., to fit the report, and the following data is shown on them: The outline of the street, the assessment limit, and all the lots within that area, each with a consecutive number, their dimensions, tax-map numbers and names. The maps are then bound in a book, with the amounts of awards and assessments shown in a tabulated report, and filed at the places of record as provided by law.

ORIGINAL IMPROVEMENT MAPS

These maps are made for the purpose of letting contracts by the Bureau of Highways for regulating, grading, setting curb, laying sidewalks and paving of streets in the Borough of Brooklyn, having been authorized by the Board of Estimate and Apportionment, and are prepared in the following manner: A survey and cross-section is made of the street to be improved, locating all existing improvements such as curbs, walks and pavements (with a note of their conditions), sewer-basins and manholes; and the necessary levels are taken for the computation of excavation and embankment, and to determine whether the existing improvements are to legal grade. The levels are started from one known bench and checked upon another.

PROFILES—This information is then plotted on profile paper of sufficient size to allow room for computations. The scale used in plotting is 40 ft. horizontal and 10 ft. vertical to 1 in., the plan showing the official street lines and widths, and the profile showing the legal grades and a three-line surface profile (center and building lines) as determined from the level notes. The quantities of excavation and embankment are computed by the end area method to the finished legal grade of the street, or if pavement is to be included, to the proper subgrade.

EXISTING IMPROVEMENTS—In computing the amount of new curb and walk to be furnished, the existing curb and walk is examined as to grade, line and condition. If found to be to the correct grade and line, in a good state of preservation and according to specification requirements, the improvements are held. If they are found to meet the last two requirements, but are not constructed on legal grade, and there is a sufficient amount to be considered, it is sometimes deemed advisable and economical to propose a change in the legal grade of the street, in order to hold these improvements. Sewer-basins and manholes, as constructed, are also tested for line and grade to determine whether they should be reset or rebuilt. After all computations are made and checked a tracing is made to accompany the specifications for letting the contract.

This plan is very complete as to details, showing the work to be done, as authorized by the Board of Estimate and Apportionment; the approximate quantities; the existing improvements, which are to be retained, are shown in dotted lines and improvements to be constructed in full lines; sewer-basins and manholes are indicated whether to be held, reset or rebuilt, and the three-line profile shows the original surface, with elevations, in different colors indicating center and both building lines. This enables the contractor to check the quantities as computed before he bids on the work. After the contract is let and the contractor is ready to proceed with his work, this bureau furnishes the grade or curb stakes, and after the completion of the work the final estimate of the excavation and embankment.

The average cost of preparing the above plans and profile including fieldwork, grade and curb stakes and final estimates of earth quantities is 9c. per lin. ft. of street.

The Cost of Land for Parkways and Parks in the District of Columbia is to be assessed hereafter in accordance with an act passed recently by Congress:

"Hereafter the United States shall not bear any part of the cost of the acquisition of land for street extensions, but when the condemnation of any land for such purposes is authorized by law the total cost of the land and the expense of the condemnation proceedings shall be assessed as benefits; in any case where land is condemned for a parkway, including street or streets, where such parkway is of considerable length with relation to its width, not less than one-half of the cost of the land, including the same fraction of the expenses of the condemnation proceedings, shall be assessed as benefits, and in any case where land is condemned for a public park not less than one-third of the cost of the land, including the same fraction of the expenses of the condemnation proceedings, shall be assessed as benefits.

Presumably the remainder of the cost is to be borne by the District.

The Reversed Dam; A Hollow Concrete Dam of a New Type

By Frank C. Osborn*

[Copyrighted, 1912, by Frank C. Osborn.]

A patent has been issued to R. C. Beardsley, of Cleveland, Ohio, on what he terms the "reversed dam." Although a structure of this type has never been built, the theory of its design and conditions of stability have been thoroughly investigated and it is expected that they will be tried out soon in practice. A brief description of the design and of the stresses in a typical structure should be of interest.

The "reversed dam" is of the general shape and construction as the now familiar hollow reinforced-concrete dam, composed of a slab deck supported by ribs or buttresses spaced at intervals and resting on a slab base. Instead of taking the water pressure against the top side of the deck, as does the hollow reinforced-concrete dam, the "reversed dam" is turned around and takes the water pressure against the under side of the deck. The hollow is thus filled with water which acts as a load to increase the stability of the dam. Below are given the computations for the typical dam of this design shown in the accompanying cut.

An example will be assumed of a dam 100 ft. high from top of base to crest, a base length of 147 ft., the inner surface of deck placed at an angle of 45° and the 12-in. ribs 15 ft. c. to c. The primary calculations assume an overflow depth of water of 4 ft., but no upward pressure under the base due to seepage or back water. A cutoff wall at upstream edge of base, together with a proper drainage system, neither shown in the drawing, should provide for the seepage water, and where conditions require provision for back-water pressure the calculations can be readily extended accordingly, as shown at the end of this article. The effect of the flowing water on upper surface of deck is comparatively small, and has been omitted.

The loads acting on this dam are: (a) The weight of dam; (b) the weight of the inclosed water; (c) the upward pressure of the inclosed water against the under side of the deck, and (d), the horizontal pressure of the inclosed water against the under side of the deck. These loads, and their moment arms around the intersection of the base with the line *OO* are as follows:

A.—Weight of Dam.

Volume of Crest	41.0 cu.ft.	× 107.8	= 4,419.8
Volume of Deck	289.0 cu.ft.	× 41.7	= 12,051.3
Volume of Base	478.0 cu.ft.	× 73.5	= 35,130.0
Volume of Rib	508.6 cu.ft.	× 87.3	= 44,394.0
	1316.6 cu.ft.	× 72.92	= 95,995.1

1316.6 × 150 lb. per cu.ft. = 197,500 lb. = weight of dam and its center of gravity is distant horizontally 72.92 ft. from line 00.

B.—Weight of Inclosed Water.

104 × 62.5 = 6500 lb. per ft. = unit pressure on base.
6500 × 143 = 929,500 lb. = total weight, and its c.g. is 75.50 ft. from OO.

C.—Upward Pressure of Inclosed Water.

The upward pressure on the deck diminishes uniformly from 6500 lb. per sq.ft. at the toe to 575 lb. per sq.ft. at the junction of deck and crest. The total amount of this pressure is

½ (575 + 6500) × 94.8 = 335,355 *lb.*,

and its center of gravity is 38.17 ft. from line *OO*.

The net vertical pressure on the crest, i.e., the difference between the downward pressure and the upward pressure, would, if the full head existed over the entire crest, be equivalent to 62.5 times the volume of the crest, 41 cu.ft., which is 2562.5 lb. As the full head is not effective over the entire crest for downward pressure, but is effective for upward pressure, the amount given above should be somewhat larger. It will be arbitrarily assumed as 2645 lb., as absolute accuracy is difficult to obtain, and, even a considerable error in this figure would not materially affect the final result.

The total upward pressure on deck and crest will then be

335,355 + 2645 = 338,000 *lb.*,

and it may be considered as acting at a distance of 38.71 ft. from line OO, determined as follows:

335,355 × 38.17 (distance from OO)	= 12,800,500
2,645 × 107.80 (distance from OO)	= 285,131
338,000 × 38.71 (distance from OO)	= 13,085,631

D.—Horizontal Water Pressure.

This equals

$$\frac{62.5\,H}{2}(H + 2h)$$

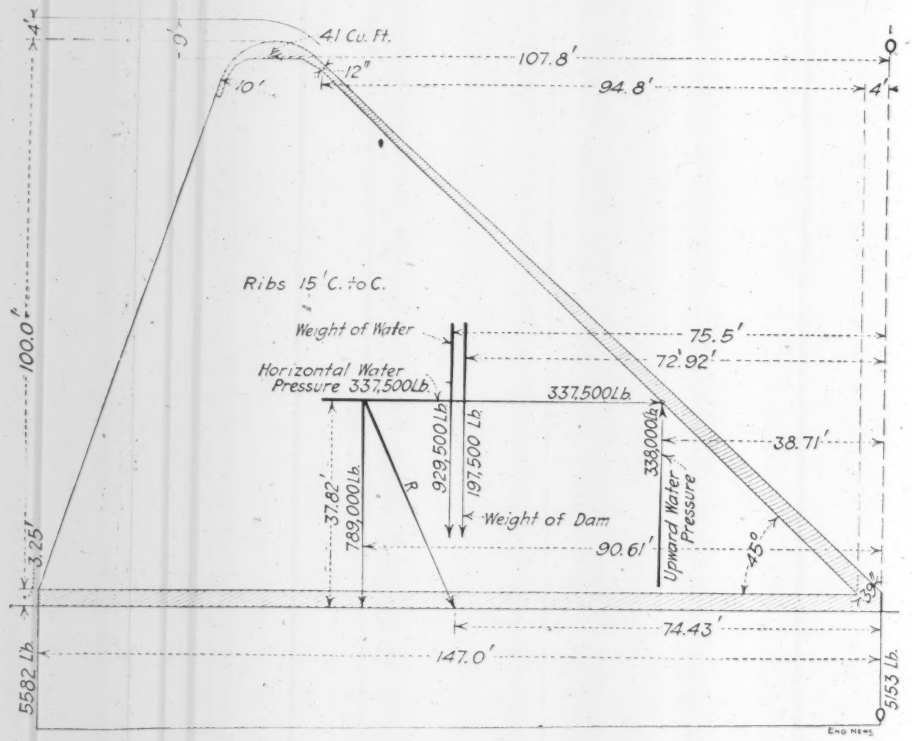
where

H = 100 ft.;
h = 4 ft.

The pressure is, then, 337,500 lb., and it acts at a distance above the base of 37.82 ft.

Footing Pressures—Taking moments now about the intersection of the base and line OO.

Dam	197,500	× 72.92 = 14,401,700	
Water	929,500	× 75.50 = 70,177,250	
	1,127,000		84,578,950
Water	338,000	× 38.71 = 13,085,600	
Water	337,500	× 37.82 = 12,764,250	
			25,849,850
	789,000	× 74.43	58,729,100



Stresses in a Typical 100-ft. Dam of the Reversed Type

Then, by the well known formula:

$$\text{Pressure at heel} = \frac{2H}{L}\left[\frac{3x}{L} - 1\right]$$

$$= \frac{2 \times 789{,}000}{147}\left[\frac{3 \times 74.43}{147} - 1\right]$$

$$= 10{,}735\,(1.52 - 1) = 5582\ lb.\ per\ sq.ft.$$

$$\text{Pressure at toe} = 10{,}735 - 5582$$

$$= 5153\ lb.\ per\ sq.ft.$$

Overturning—The factor of safety against overturning is readily obtained by dividing the resultant moment of stability by the resultant overturning moment.

84,578,950 ÷ 25,849,850 = 3.27.

Sliding—The factor of safety against sliding is obtained by multiplying the net vertical pressure on foundation by the coefficient of friction (assumed at 0.65) and dividing the product by the horizontal water pressure.

789,000 × 0.65 ÷ 337,500 = 1.52.

*Consulting Engineer, Engineers Building, Cleveland, Ohio.

BACKWATER PRESSURE

The effect of a backwater pressure due to a height of 30 ft. above the bottom of the base may be computed as follows:

The pressure under the base is

$$30 \times 62.5 = 1875 \text{ lb. per sq.ft.},$$

and the total upward pressure is

$$1875 \times 147 = 275{,}620 \text{ lb.}$$

The downward pressure on the toe is 22,565 lb. and the horizontal upstream pressure is 28,125 lb.

Taking moments about the intersection of the base and line OO we obtain,

275,600	× 73.5 =		20,256,600
22,565	× 9.0 =	203,085	
28,125	× 10.0 =	281,250	484,335
253,035	× 78.14 =		19,772,265

from which the foundation pressure due to back water may be obtained and then combined with those previously found.

$$\text{Pressure at heel} = \frac{2W}{L}\left(\frac{3x}{L} - 1\right)$$

$$= 3442\,(1.5945 - 1) = 2046 \text{ lb. per sq.ft.}$$

$$\text{Pressure at toe} = 3442 - 2046 = 1396 \text{ lb. per sq.ft.}$$

Combining these pressures with those found above,

$$\text{Pressure at heel} = 5582 - 2046 = 3536 \text{ lb. per sq.ft.}$$

$$\text{Pressure at toe} = 5151 - 1396 = 3757 \text{ lb. per sq.ft.}$$

OVERTURNING—The factor of safety against overturning, backwater pressures included, is obtained by dividing the resultant moment of stability by the resultant overturning moment.

The resultant moment of stability is

$$84{,}578{,}950 + 484{,}335 = 85{,}063{,}285 \text{ ft.lb.}$$

The resultant overturning moment is

$$25{,}849{,}850 + 20{,}256{,}600 = 46{,}106{,}450 \text{ ft.lb.}$$

And the factor of safety is

$$85{,}063{,}285 \div 46{,}106{,}450 = 1.86$$

REVERSED VS. BUTTRESSED TYPE

A comparison of the calculations for overturning for the "reversed" type with those for the same dam used as the buttressed type shows a very interesting and paradoxical result.

Used as the buttressed type the load on the deck is

$$\tfrac{1}{2}\,(6500) \times 107.8 = 350{,}300 \text{ lb.},$$

which acts at a distance of 111.07 ft. from the toe.

The weight of the dam is 197,500 lb., and acts at a distance of 74.08 ft. from the toe.

The resultant moment of stability is then,

350,300	× 111.07 =	38,907,800
197,500	× 74.08 =	14,630,800
		53,538,600

The overturning moment is 12,764,250.

The comparison then shows as follows:

	Buttressed Type	Reversed Type
Moment of stability	53,538,600	84,578,950
Overturning moment	12,764,250	25,849,850
Factor of safety by division	4.19	3.27
Comparative safety by subtraction	40,774,375	58,729,100
Dividing by 103.25	394,960	568,800
Comparison by percentage	100	144

The above table shows that while the "reversed dam" has a *smaller* factor of safety, when computed in the customary manner, it has actually a *greater* margin of safety against overturning by 44%.

Expressed in another way, the buttressed design would just withstand a force of 394,900 lb. applied horizontally at the crest, as ice pressure would be applied, while the reversed design would similarly withstand a pressure of 568,800 lb.

It is also interesting to note that the sliding resistances compare in exactly the same ratio.

	Buttressed Type	Reversed Type
Weight of dam	197,500	197,500
Net downward pressure of water	350,300	591,500
	547,800	789,000
Multiplying by 0.65	356,070	512,850
Comparison by percentage	100	144

The total foundation pressures will compare, of course, in the same ratio and as the center of pressure is located at the same distance from the center of the base, 0.93 ft. in each case, it follows that the unit pressures at toe and heel will also compare in the same ratio as is shown by the following:

Dam	197,500	× 72.92 = 14,401,700
Water	350,300	× 35.93 = 12,586,300
Water	337,500	× 37.82 = 12,764,250
	547,800	× 72.57 = 39,752,250

$$\text{Pressure at toe} = \frac{2W}{L}\left(\frac{3x}{L} - 1\right)$$

$$= \frac{2 \times 547{,}800}{147}\left(\frac{3 \times 72.57}{147} - 1\right)$$

$$= 7452\,(1.48 - 1) = 3577 \text{ lb. per sq.ft.}$$

$$\text{Pressure at heel} = 7452 - 3577 = 3875.$$

	Buttressed Type	Reversed Type
Pressure at toe	3,577	5,153
Pressure at heel	3,875	5,582
Comparison by percentage	100	144

An Opportunity for a Bridge Across the North River at New York City*

BY GUSTAV LINDENTHAL†

The present agitation for better freight transportation and dock facilities in New York has an important bearing upon another momentous question, and that is the bridging of the North River.

The bridge project is very old and it is not necessary here to go into the history of all the attempts for its realization. Its great cost, ranging from $70,000,000 to $100,000,000 (including the costly right-of-way), according to location and capacity, has been the principal deterrent to all efforts heretofore.

The subject comes up now in a peculiar manner.

The City of New York requires urgently more water front on the Manhattan side for steamship piers and freight collection and distribution facilities. The only way to get additional water front is by inducing all the railroads having freight landings and piers in Manhattan to abandon them. They occupy in the aggregate nearly 4000 ft. between the Battery and Sixty-fifth St. They now bring the freight across and also up and down the river on car floats. The present Commissioner of Docks and Ferries in New York contends that the railroads should build new modern freight terminals in Manhattan back of the river front, and abandon their present antiquated system, which deprives the city of valuable room needed for more steamer piers.

To that end the Dock Department has prepared tentative plans showing an elevated freight railroad along the river front from Sixty-fifth St. down to the Battery, with spurs into freight terminals located along this railroad, and other spurs to be run directly to the steamship piers.

The plans further provide for this elevated railroad to connect through tunnels under the North River and Bergen Hill with a large classification and distribution freight yard in the meadows of New Jersey, into which all the railroads will run their tracks. In this manner freight cars from all these roads could be brought directly into New York into the new freight terminals. There would also be markets for foodstuffs in the terminals below the tracks on the street level, so much needed for public convenience and economy.

The plan is comprehensive and would solve not only the Manhattan freight problem for the New Jersey railroads, but would also be largely helpful in the speedier loading and unloading of freight from and to steamships. It is proposed that the elevated railroad be built by the city and that it be used also by the New York Central R.R. from Sixty-fifth St. down. That would permit the present tracks of that railroad on the streets to be everywhere taken up.

The scheme would, undoubtedly, be of great benefit to the public, but it means to the railroads a very large expenditure. A large percentage of the total cost would be for the tunnels under Bergen Hill and the North River, each about 11,000 ft. long. Steep grades would be required in them to rise from an elevation of about 90 ft. below water to 50 ft. above, or a total of 140 ft., resulting in grades of 4 to 5 per cent., which is very steep for railroad operation, and much steeper than it would be over a bridge.

The railroads in New Jersey feel very reluctant to assume the great financial burden which that plan involves, but the city may compel them to give the terminal question serious consideration. Recent legislation empowers the city to condemn waterfront property; also prop-

*From the "New York Times" of Dec. 9.

†Consulting Engineer, 68 William St., New York City.

erty, owned by the railroads in fee. Some of the piers, long owned by the city, are leased to the railroads. When the existing leases expire, the city grants to the railroads only revocable permits for the use of such piers. The city can, therefore, at any time, force the railroads off its water front, but such a step would hardly be taken rashly, since the public would suffer as much injury as the railroads from such an arbitrary proceeding.

The hesitation of the railroads is easily understood, not only by reason of the large expenditures facing them, but also because of the larger amount of taxes that would have to be paid by them on freight terminals in New York. The question whether sufficient net revenue to cover interest and taxes on all that new investment can be earned is a most serious one for the railroads.

If the only problem the city authorities had to solve was the freight traffic, then it might be a matter of indifference whether the railroad connection across the river be by tunnels or by bridge; it must be one or the other, as freight transfer by car floats, which is the present system, has become inadequate and unsatisfactory. The railroads when they act will doubtless prefer tunnels to the bridge, notwithstanding the steeper grades, because of their apparent lower cost of construction for their own particular purposes.

But is this the view that the city and the adjoining states will or should take? There is a great public demand for a bridge, particularly since the automobile traffic came into being. Automobiles can now cross the North River only on ferries, requiring from twenty to forty minutes (according to waiting time), while on a bridge they could cross in three minutes.

Then there are the larger functions of such a bridge for all kinds of traffic, and its greater availability. It would effectually settle the questions of the isolation of New York City from the remainder of the continent, and the direct fusing of this part of the metropolitan area with the rest of the city for its benefit.

If the railroads should decide for freight tunnels, then the idea of a bridge crossing the Hudson River may have to be abandoned for a long time, perhaps forever. Because a bridge, without the railroad traffic, could never earn enough from the local transit lines and from the automobile and motor-truck traffic, dense as both kinds of traffic would be over it, to pay interest on the investment. But careful estimates show that a bridge built for the combined traffic of the railroads, local transit lines, automobiles, and wagons, and pedestrians, would after a few years produce more than sufficient revenue to meet the interest and sinking fund for its construction. For the beginning it may be necessary, however, that a part of the funds be obtained (to be later reimbursed) from the increment value of real estate within a certain radius on the New Jersey side of the bridge, or that the real-estate interests should join with the transportation companies in furnishing part of the capital required.

That real estate could not fail to appreciate in value from two to ten times its present value. What is needed, therefore, is that the appropriate interests on the New Jersey side, through representatives, should come into a conference with representatives of the New Jersey railroads for the purpose of inducing them to build a bridge instead of tunnels and evolving a plan under which the undertaking could be jointly financed.

Of all the New Jersey railroads the Pennsylvania R.R. has more than 50 per cent. of the traffic crossing the Hudson River to Manhattan. That company, therefore, would be an important factor in the creation not only of the new freight tunnel facilities in Manhattan, but in the eventual construction of a bridge across the North River. The fact that the Pennsylvania R.R. already crosses the river through its own tunnels to its own terminal in Manhattan exclusively used for its passenger traffic, and many of the other lines by transfer send their passengers through the Hudson Tunnels, narrows the problem down to freight facilities in the first place. Later, however, the other railroads in New Jersey may desire to cross the bridge into Manhattan with their passenger trains. But the passenger terminal station required for such a plan would have to be a separate financial undertaking.

The bearing of the plans of the Commissioner of Docks and Ferries for better port facilities in Manhattan upon the possibility of a bridge across the North River may thus be seen to grow out of an unprecedented situation which will not repeat itself, and which opportunity should not be missed. The legal and financial difficulties of such a large undertaking can be readily overcome by the coöperation of the parties most interested, and we may be assured that capital is ready to step in if that is done.

New York is now virtually cut off from the continent by the North River as regards all street and road traffic, including automobile and motor-truck traffic. Long tunnels are useless for them, because the poisonous gases could not be removed by ventilation quickly enough to prevent suffocation. Electrical motive power alone could be used through them, but that will not likely be generally used for street-motor transportation because of its greater cost.

There are already six tunnels under the North River (two for part of the passenger traffic of the Pennsylvania R.R. and four for the local passenger traffic of the Hudson and Manhattan system). They are all below Thirty-fourth St., Manhattan. But to connect the high plateaus on each side of the river north of Fiftieth St., tunnels for street purposes are impracticable, because of the unworkable steep grades that would be required to dive from high ground 100 to 180 ft. above to 90 ft. below the water surface.

No one questions the entire practicability of a large bridge over the Hudson River. Some twenty years ago much doubt existed on that point, even among some engineers, but no such doubts exist now. Given the funds, a bridge of a single span at the proper height and over the entire river between pierhead lines, can surely be built.

The bridge cannot be financed and built by the two states, or otherwise with public funds. The restrictions upon the use of public credit contained in the Constitutions of both states—New York and New Jersey—make it impossible. This point has been most carefully investigated by legal authorities. The bridge can be built only with private capital.

The large plateau on the New Jersey side—nearly 20 miles long and averaging a mile wide—is an ideal place for a large city, now almost undeveloped and accessible only after hours of slow ferry and car traffic. A bridge would produce there another Brooklyn within the metropolitan area and dependent on Manhattan as a business and supply center. Looking into the future, the bridge would thus create populous and wealthy communities. Additional bridges would follow in due time, as they did over the East River. But the construction of the first bridge is feasible only by a combination of the different interests as indicated.

It would therefore seem wise at the present juncture, when dock plans and the relief of the river front and the streets of lower Manhattan are being considered, and above all the maintenance of New York City as a harbor and port are directly concerned, that the bridge project as an adequate means of relief to all kinds of traffic, and as a comprehensive project to unquestionably insure the continuance of New York as a great port, should have serious consideration.

When the great usefulness of a bridge in providing relief for every kind of transit and in connection with warehouses, yards, electrification, etc., and the connection over it of all lines, passenger as well as freight, both outside and inside New York, is taken into account, it appears rather shortsighted to imagine that the only serious question involved in the present situation is relief at the river front in a small portion of Manhattan.

As a physical evidence of what a bridge can accomplish for New York, just add to the facilities of the bridge connecting Manhattan, Brooklyn, and Queens a connecting line with reasonable grades bringing trunk railroads into New York City, and relieving that city of its isolation from the rest of the continent.

The Sanitary Control of the Ruhr River, Germany

By Richard H. Gould*

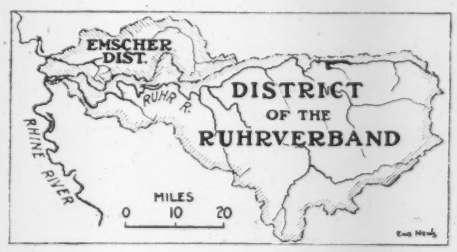
In Germany the sanitary control of rivers and the handling of all sewage in their watersheds through a single organization seems to be growing in favor. The Emschergenossenschaft† was the first organization of this type to be formed in Germany, or in fact in the world, and in the eight years since its founding has proved so satisfactory that other similar organizations have been created, and are being proposed, only slightly different in form from the pioneer. The newest of these sanitary organizations, the proposed "Ruhrverband," is to control the Ruhr River, which drains the watershed just south of the Emscher River (see accompanying sketch map).

The Ruhr is in northwestern Germany and flows into the Rhine at Duisburg. It has a mean flow of 3060 cu.ft. per sec. and drains a watershed 80 miles long and 1740 sq.mi. in area. In the upper reaches the district is either wooded or given over chiefly to agriculture, and is but sparsely settled. Farther down there are industrial towns, mines and other varied interests. Near the mouth of the stream are the large industrial and trading cities of Mulheim and Duisburg. There are 1,160,000 people living in the district, the larger portion of them being in the lower and central portions.

The water of the Ruhr is comparatively soft and furnishes almost the entire water-supply of its valley as well as that of the adjoining Emscher district. There are in fact 89 water-works in operation, which in all withdraw on an average about 350 cu.ft. of water per second. All of these collect their water by the same method, namely, by natural filtration through the river banks into lines of driven wells and galleries placed in the gravel of which the banks are composed. The supply is dependent upon the quantity of water which will filter through the river banks, and at times of low water the limit has in many instances been reached. In such cases the large amounts of sludge which the river carries form a nearly impenetrable cake on the river bottom, and the water-works are then short of water until the sludge cake is washed away by the next high water.

The growth of population and industry in the Ruhr district has been rapid in recent years and there has been a corresponding increase in the pollution of the river, thus making it very urgent, especially in view of the drinking-water situation, that steps be taken to rectify existing conditions and control future developments.

At the request of Regierungspraesident Von Bake of Arnsberg, in whose district the Ruhr largely lies, the conditions were investigated by Dr. Ing. Karl Imhoff and the results of the investigations and the recommendations based on them printed in a small report called "Die Reinhaltung der Ruhr."* The chief points brought out here were: (1) that large quantities of untreated sewage were entering the river where in summer the dilution was insufficient; (2) that the dilution was decreased by the artificial concentration due to the withdrawal of water for drinking purposes; (3) that large amounts of sludge in the water interfered seriously with the safe operation of water-works taking their supply from the river; and (4) that for reasons aside from those having to do with the water-supply question immediate steps for betterment should be taken. The recommendations were (1) that all sludge be kept out of the river as far as possible, and (2)



that, as the cheapest and most desirable method for bringing about this end, a federation be formed similar to the Emschergenossenschaft for the purpose of handling the sewage of the entire valley.

The water-works had already united to build reservoirs in order to store flood water and distribute it more evenly throughout the year, and these works, so far as they are completed, aid in giving a larger dry-weather flow and thus help the operation of the water-works. The matter in hand, however, did not concern the water-works alone and from its nature was not to be handled by an organization consisting of one special interest. The existing governmental divisions were also not suitable and so steps were taken to bring about the organization recommended. With this in view a meeting of all the cities, towns and industrial and other interests in the valley was called and a temporary organization, the "Ruhrausschuss," formed to draw up a law under which the proposed new federation should be formed. After deliberation this body proposed a law, or constitution, the most important points of which, according to Landrat Gerstein,† chairman of the "Emschergenossenschaft" and "Ruhrausschuss," are the following:

(1) The purpose of the federation is to prevent pollution and to better the condition of the Ruhr River.

(2) The methods by which this end is to be gained are not defined, but will consist chiefly of building and operating works for purifying and removing the sewage.

(3) The members of the federation having power to direct its operations are the towns and cities in the valley and the water-works federation.

(4) Those benefitting by the works of the federation or those causing it expense, these being chiefly the communities, industrial plants, and the water-works, will be assessed for the costs incurred by the federation.

(5) The cost of the sewage-purification plants will be borne by the members in common, the water-works paying one-third of the entire cost.

(6) The amount of an assessment depends upon the extent to which the assessed party is responsible for the pollution, and upon the amount of benefit to be received by it from the federation works.

(7) Disputes as to the distribution of assessments are not to be decided by ordinary legal procedure, but by a special body elected by the federation.

This proposed law has been accepted by the provinces of Rhine and Westphalia and is now before the Prussian legislative body.

While waiting for the acceptance of the law outlined above the temporary organization, the "Ruhrausschuss," has taken up the actual work of the federation. Dr. Imhoff has been appointed chief engineer and arrangements have been made whereby the officials, engineers, and facilities of the Emschergenossenschaft are available for carrying out the necessary works. The operation of existing sewage-disposal works has been taken over and several large new works are now in construction. In addition to this, special methods of purification that may have application to the new district are being worked out in the experimental stations of the Emschergenossenschaft.

The principal works of the proposed "Ruhrverband" will probably be finished in a few years and its duties will then be chiefly the operation of the sewage-disposal works, with an occasional extension or new installation. With the proposed works in operation most of the sludge will be kept out of the river, and, as now thought desirable, the sewage entering the river will be purified beyond the point obtainable by simple settling action. The sludge deposits on the river bottom will thus be prevented, the condition of the water bettered, and, due to increased reservoir capacity to be supplied by the water-works organization, the summer flow of the river will be increased. A supply of good drinking water, abundant for any demand that may be expected, will thus be secured, and the Ruhr kept in such a state of purity as to be suitable for recreation and industrial purposes.

*Assistant Engineer with Emschergenossenschaft, Essen, Germany.

†See Eng. News, Dec. 1, 1910.

*Published by C. W. Haafeld, Essen, Germany.

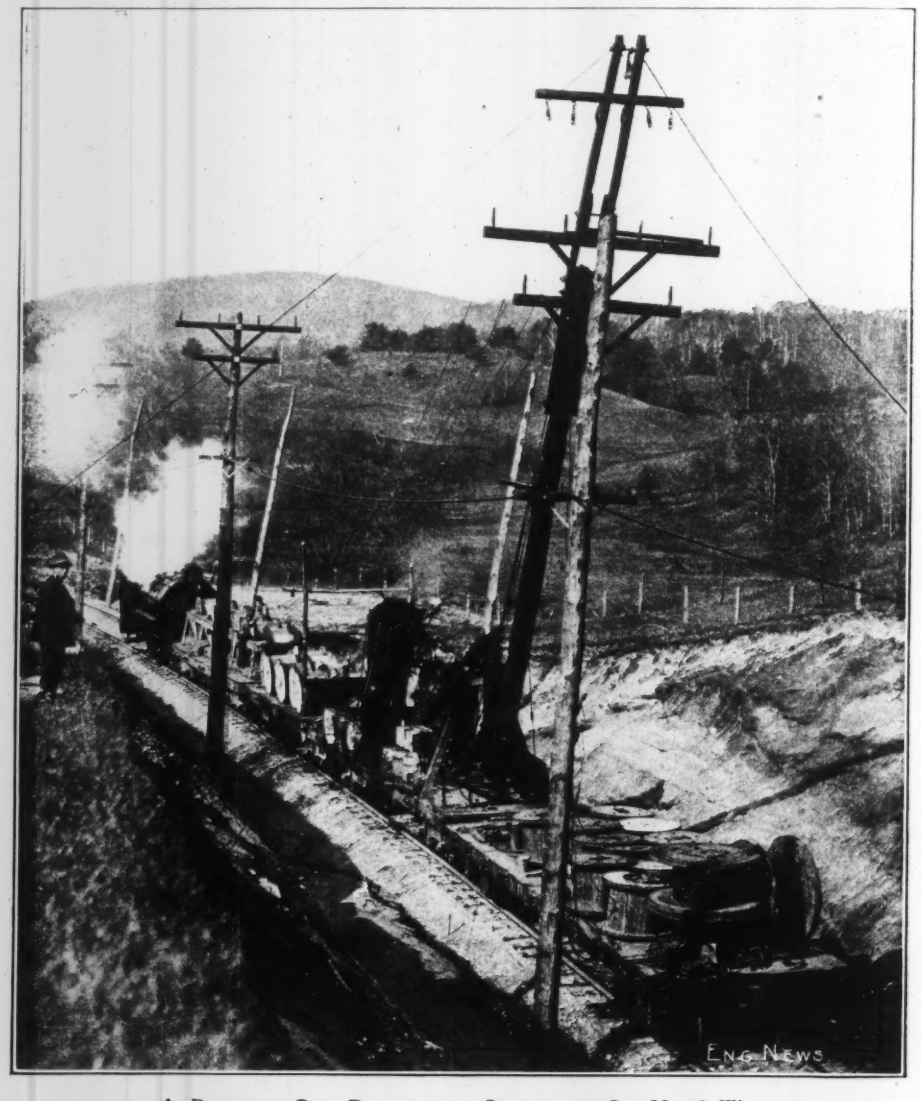
†Gerstein, Der Ruhrverband in Essen.

FIELD AND OFFICE

A Derrick Car for Stringing Six Wires at Once

The Western Massachusetts Contracting Co., in building a trolley line from Lee to Westfield, in Massachusetts, strung all the wire with a derrick car. This was rigged to raise and place as many as six wires at one time. The accompanying figure shows six No. 0 wires being placed directly on top of the transmission poles.

The wire reels were mounted in a large stand and each was fitted with a band brake, so that they did not require any attention. Four men with the outfit shown can run from 18 to 24 miles per day, it is claimed. Frank B. Lee, of Lee, Mass., was superintendent of construction



A DERRICK CAR RIGGED FOR STRINGING SIX NO. 0 WIRES
(Western Massachusetts Contracting Co.)

A Weed-Killing Train

The killing of weeds and grass along railway tracks by means of a chemical solution or a hot flame from a burner is practised extensively by many railways, and for the former process a complete weed-killing train is operated by the Interstate Chemical Co., of Galveston, Tex., when it has large contracts from individual railways. The sprinkling car carries an 8000-gal. tank and has two centrifugal pumps driven by a 25-hp. gasoline engine; the solution (known as Dinamine) is distributed over the track by six sprinkling heads, four at the head and two at the rear end of the car, each discharging about 250 gal. per min. under pressure. The discharge is regulated by valves, etc., so that the solution can be sprayed over a width of 1 ft. to 150 ft. outside of the rail; it can be sprinkled between the rails only, or one side of the track, or only outside of the track, as may be required.

The train consists of the sprinkling car, three to six tank cars of 10,000-gal. capacity, a tool car, and a locomotive and caboose. The tool car has an engine, centrifugal pump, pipe lines and other equipment for diluting the concentrated chemical with water from the tank cars and delivering the solution to the tank on the sprinkler car. The sprinkling crew consists of 2 enginemen, 4 pipemen and 2 helpers. The railway company furnishes the train equipment and train crew and supplies the water. The train is operated as a special work train, under the ordinary railway rules, and is run at a speed of 5 to 15 m.p.h. The equipment is sufficient for a run of 30 to 40 miles without a stop for reloading.

For smaller orders, the chemical company furnishes a sprinkler outfit which can be applied to an ordinary tank car. A vertical pipe from the bottom of the tank carries a 1½-in. horizontal pipe 6 ft. long, extending across the track; each end of this has a ball-joint connection for a 3-ft. length of pipe, the outer end of which is closed. The pipes are perforated and the outer portions can be swung aside to clear obstructions. The main valve is placed in the vertical pipe.

The process has been operated by this company since 1902. It has been used regularly on the Gulf, Colorado & Santa Fé Ry. (A., T. & S. F. Ry. System) since 1904, and this road has already placed contracts for next year's work. It has been used on the Fort Worth & Denver City Ry., the Chicago Great Western Ry., the Panama R.R., the Panama Canal, and the Madeira & Mamore R.R., in Brazil.

It is stated that in the semi-tropic districts of the Southwest the cost of weeding track by hand labor (pulling weeds by hand or cutting them close with a shovel) in rock and gravel ballast, for a width of 8 ft., averages $67.25 per mile; and this work often has to be done two or three times in one season.* The chemical sprinkling system costs about $67.50 per mile, and the number of applications depends upon the number of times the grass is cut. Its great advantage is that it kills the roots and vegetation and saturates the ground so that a new growth will not come for about five years.

*In Tratman's "Railway Track and Track Work," the cost of cutting grass and weeds in earth ballast (but cutting only along each side of each rail) is given as $7.50 per mile. This work may have to be done three or four times in a season. If the cutting is across the full width of the ballast the cost will be about $12.50 per mile, but this may be required only once or twice a year. The total annual cost is given as $35 to $50 per mile.

It is claimed that it saves at least three years of expense for grassing or grass-cutting, and four years of the expense for weeding. Further, it does not disfurb the ballast, as in hand work. This particular chemical has been used with success on all kinds of ballast.

One of the special fields of its application is at yards and sidings, where it is almost impossible for the ordinary gangs to keep the weeds cut, in addition to their other work. Places of this kind on the A., T. & S. F. Ry. treated with the chemical solution four years ago are still clear of weeds. It is specially useful, of course, in districts where there is such persistent and rapid-growing vegetation as the Johnson grass (which is a source of endless trouble), and also in tropical countries where vegetation grows almost as fast as it is cut and cleared.

Elevating Plant for Concrete Materials

Delivery of materials for making concrete, in building the Atherton Ave. 180-ft. arch bridge over the Pittsburgh Junction R.R. for the city of Pittsburgh, Penn., is done by a set of elevators and a conveyor which took cement and sand from railway cars in the valley bottom to bins up on the bank. The two photographs herewith show how this plant is set up.

Alongside the track are placed two elevators side by side: (1) A bag elevator, 49 ft. long c. to c.; and (2) a bucket elevator, 60 ft. long c. to c. The former has wheeled bag-carriers attached to a single strand of chain. Bags of cement are delivered to it at the car floor, and discharge at the top into a chute leading to a belt conveyor. The bag elevator can handle about 13 bags per minute. The bucket elevator reaches into a pocket below the track, to take sand or gravel unloaded from drop-bottom cars. At its upper end it feeds into the same chute as the bag elevator. It can unload a car of gravel in 1 hr. and a car of sand in 2 hr.

From the discharge chute of the elevators a 16-in. conveyor belt 180 ft. long, set on an incline, extends to the storage bins near the top of the bank. This belt takes either bags from the bag elevator or bulk sand or gravel from the bucket elevator.

The three machines are driven by a single 10-hp. motor, placed at the head of the elevators, with clutch connections.

This equipment was furnished by the Link-Belt Co., Philadelphia, Penn., but was set up by the bridge contractor, the Friday Contracting Co., of Pittsburgh. It cost about $900.

A Magnetic-Load Machine for Alternating Stress Tension Tests, and some Results

It has been difficult to design a tensile-endurance testing-machine in which the load is applied with a high frequency by mechanical means, since the masses of the various parts, in combination with their unavoidable springiness, introduce inertia actions which affect the load. A machine has been built by B. P. Haigh, of Glasgow, Scotland, which employs an alternating magnetic flux for producing a pulsating pull in a wire specimen. He described it recently before the British Association for the Advancement of Science. The machine is in use in the James Watt engineering laboratories of the University of Glasgow, where it was constructed. It is intended primarily for testing wire with pulls varying up to about 100 lb. In Fig. 1 is shown the principle of its design.

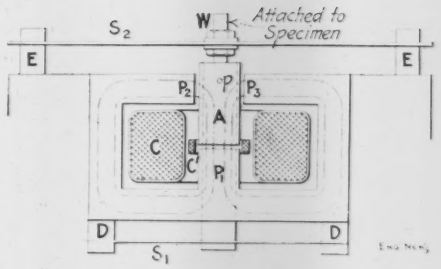
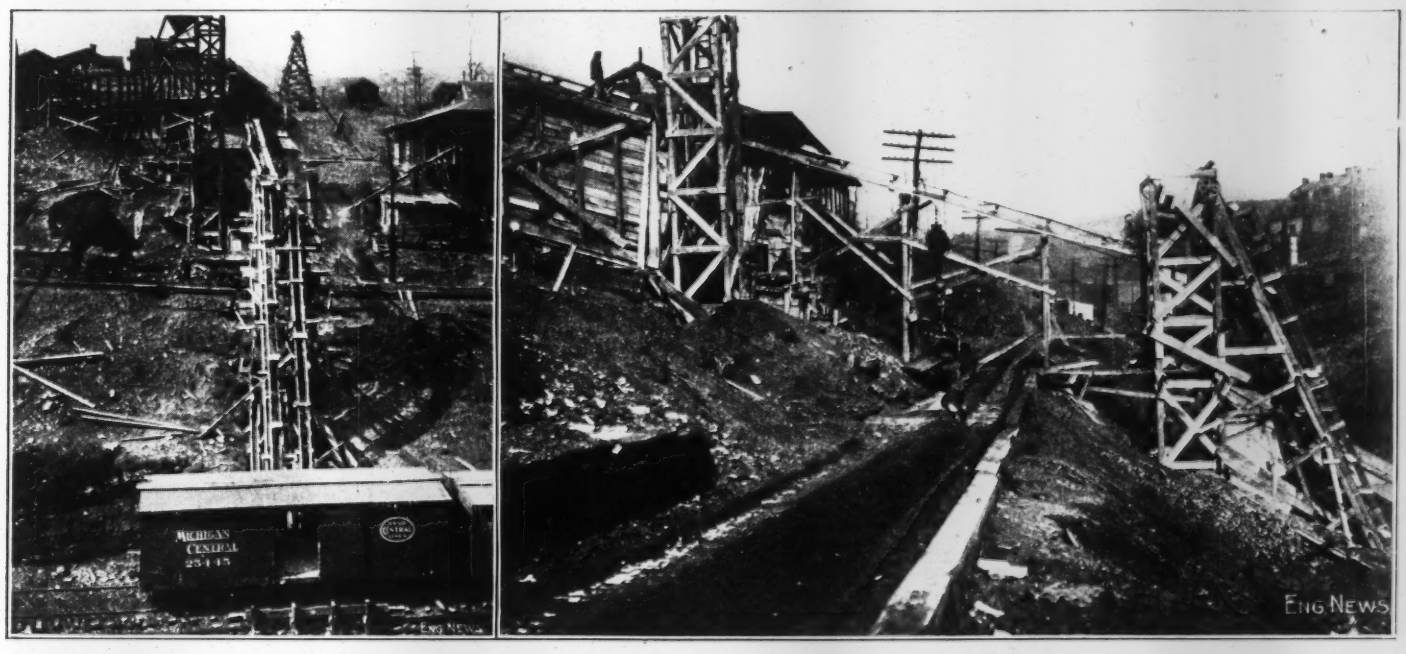


FIG. 1. DIAGRAMMATIC ARRANGEMENT OF THE HAIGH WIRE-TESTING MACHINE

The lower end of the wire specimen W is attached to an armature A, a laminated block of iron supported on flat springs, S_2 and S_1, to give freedom over a small range for vertical motion only. The armature is directly over the pole P_1 of the magnet. The magnetic flux produced by the large coil C flows through the core, pole pieces P_1, P_2, P_3, armature and the gaps. The total air gap in the circuit is made small so that a moderate-sized coil produces a strong pull.

The magnetic pull is proportional to the square of the ratio of voltage to frequency. This holds good so long as a constant proportion of the total magnetic flux passes across the gap; however, the leakage flux varies with the length of gap so that it is important to keep this constant.

The springs S_1 and S_2 are arranged primarily to guide the armature between the poles, keeping its lower face parallel to that of the pole piece P_1. By using springs instead of guides, the effect of friction is minimized; however, when working with high frequencies even the air resistance of the armature and attachments becomes a measurable quantity,



MATERIAL ELEVATING AND CONVEYING PLANT FOR CONSTRUCTION OF ATHERTON AVE. P. J. R.R. BRIDGE, PITTSBURGH, PENN.

though generally small enough to be negligible. The lower spring is of light material, like a clock spring, and serves only to keep the armature central. It is attached to the armature by bayonet strips, which pass on either side of the main pole P_1 and is held at the end by clips D on the yoke frame. The upper spring is made of heavier material, as it serves the extra purpose of compensating for the inertia forces required for accelerating and retarding the mass of the armature. The stiffness of this spring may be adjusted, to suit the test frequency, by moving the clamping saddles E inward or outward on the upper part of the frame. This upper spring S_2 is in two parts and is secured between nuts on a screw spindle borne by the armature block. By adjusting the nuts together the rest position of the armature is moved up or down, as desired. The armature is held in central position during this adjustment, by means of the pin p passing through it and the frame. By this precaution the saddles can be adjusted without drawing the armature against the poles P_1 or P_2.

It is found desirable to work with a load wave of sine shape, as this means that the pressure is carried out under a single frequency and not with a combination of harmonic frequencies superimposed on some fundamental. A small search coil for checking wave form by means of an oscillograph is wound around the pole-piece P_1 as closely as possible to the voltage coil C_1. This latter is connected directly to a sensitive voltmeter and acts like the secondary of a transformer. It is introduced so that the voltmeter reading from which the load is deduced may be as far as possible free from leakage-flux errors; the coil is wound with fine wire and is very close to the edges of the pole face, so that the greater part of the leakage flux passes outside the coil.

Standardizing the machine consists in determining the mean value of the pull of the magnet experimentally. When this is known, together with the wave shape of the magnetic flux, the maximum value of the pull is readily deduced. Thus when the flux follows a sine wave, the pull follows a sine wave of double frequency and the mean value is one-half of the maximum.

Some Test Results

Only a limited number of materials have been tested under pulsating loads in the machine. One of these was a low-carbon (0.15%) annealed wire breaking under a steady stress of about 53,000 lb. per sq.in. with 20% extension. The second material was a hard-drawn steel of higher carbon (0.45%) breaking at about 360,000 lb. per sq.in. with 1.7% extension. The results obtained from ductile material are shown in Fig. 2, in which curve A represents the autographic diagram obtained with steady load, duration of loading being 15 min. Curve B represents the extension of the same material under a pulsating load in this machine with a frequency of 62 cycles per sec. The extension begins at an earlier figure and the maximum load does not exceed 36,000 lb. per sq.in., which is close to the yield-point under steady load. The percentage extension is approximately equal for the two curves. The fractures were carefully examined and did not appear to differ noticeably from those obtained with steady load. In both cases the characteristic local extension of the ductile material was formed. The curves C and D represent diagrams obtained with pulsating load when the specimen had been previously stretched with a steady pull and at release leaving, respectively, 3 and 5% permanent extension. The curves rise steeply and afterward follow curve B. The rounding at the junction with curve B is less noticeable with 5% than with 3% initial extension.

When part of the extension had been carried out with pulsating load, released and then subsequently continued, curve E was obtained. There was no apparent rounding off of the junction of curves E and D, but extension recommenced sharply when the load reached its former value. An interval of rest of 24 hr. did not appear to alter this result. When the specimen had been extended by pulsating load and the load was then somewhat released, say 10%, to the point represented by C, it did not appear that any further extension occurred. At any rate, after a very short period had elapsed, the reduced load was applied at least 2,000,000 times without further extension and fracture occurred at the grips.

The hard-drawn wire was more difficult to grip, as its pressures were so high that slipping and fracture at grips by hammering were difficult to avoid. It appeared that the strength with the pulsating load was not greatly less than that with a steady load. The better specimens broke with a load of about 270,000 lb. per sq.in. pulsating, combined with 45,000 lb. per sq.in. continuous (applied wth the object of minimizing vibration), giving a total of about 315,000 lb. per sq.in. compared with 360,000 tons per sq.in. obtained in tests with steady loads.

No clear evidence of fatigue was found in either ductile or hard-drawn material.

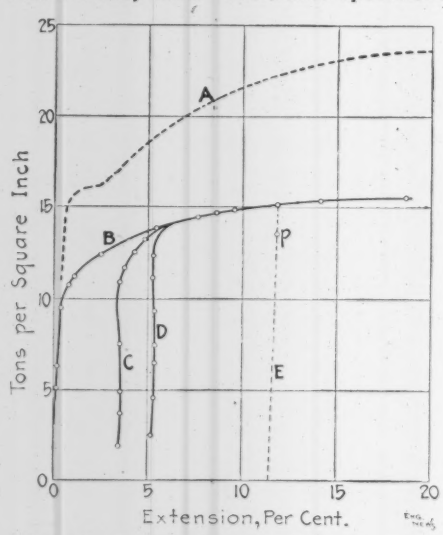


FIG. 2. STRESS-STRAIN CURVES FOR ANNEALED IRON WIRE

More on Wing-wall Problems

By C. M. Luther*

In Engineering News of Nov. 21, 1912, G. L. Jensen presents formulas for wing abutments. The writer expected to read further discussions by engineers of varied experience in designing different types of wing abutment. The silence of more capable designers is his excuse for bringing forth a few points concerning the subject.

If the necessary data for the designing of a wing abutment are complete and accurate at the start, the designer should calculate all the essential dimensions and angles and show them clearly on the plan, so that the engineer in the field can lay out the abutment without himself being compelled to make any further computations. It must be borne in mind that it is the engineer in the field (whether he is a capable man or not) who is most likely to criticize the plan and find fault with it—one very good reason why all the dimensions and angles should be accurate.

Front Batter to be Found

Mr. Jensen's formula to compute the angle ø assumes that the batter on the front face of the wing-wall is given; therefore it is applicable only to the type of abutment shown by his sketch. But the front batter of the wing-wall cannot always (or should not always) be chosen at the pleasure of the designer. Under certain conditions it happens frequently that, there being given the height of the abutment, the angle θ and the slope of the wing-wall in a vertical plane perpendicular to the center-line of the track, the point D or the distance BD in plan in Fig. 1 determines the angle ø and also the front batter of the wing. In Fig. 1 the toe of this batter hits the breast of the abutment at B, a point conveniently chosen and a certain known distance from A. The vertical projection $B'D'$ of this batter, on a plane along AD, is made to pass through such a point that the portion of the wing batter above the level of the bridge-seat falls entirely to the back of the face of the ballast-wall, thus avoiding a vertical triangular surface in the plane of AD and extending forward from the face of the ballast wall.

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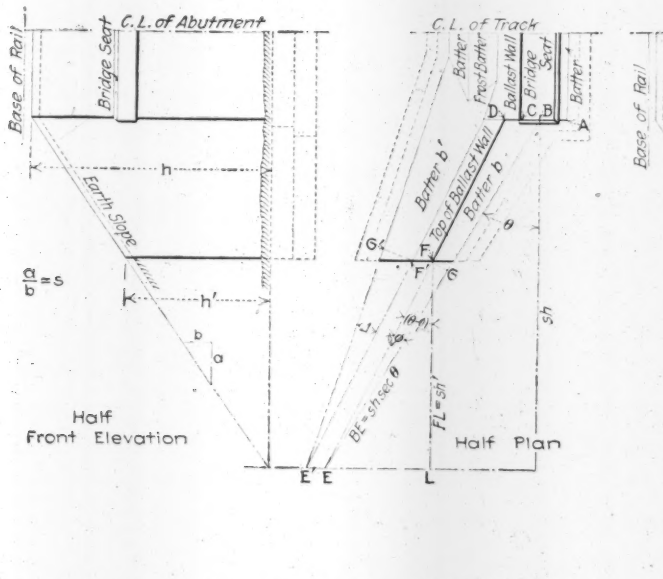

FIG. 1. SKETCH OF WING ABUTMENT FOR DECK SPAN; FRONT BATTER COMPUTED

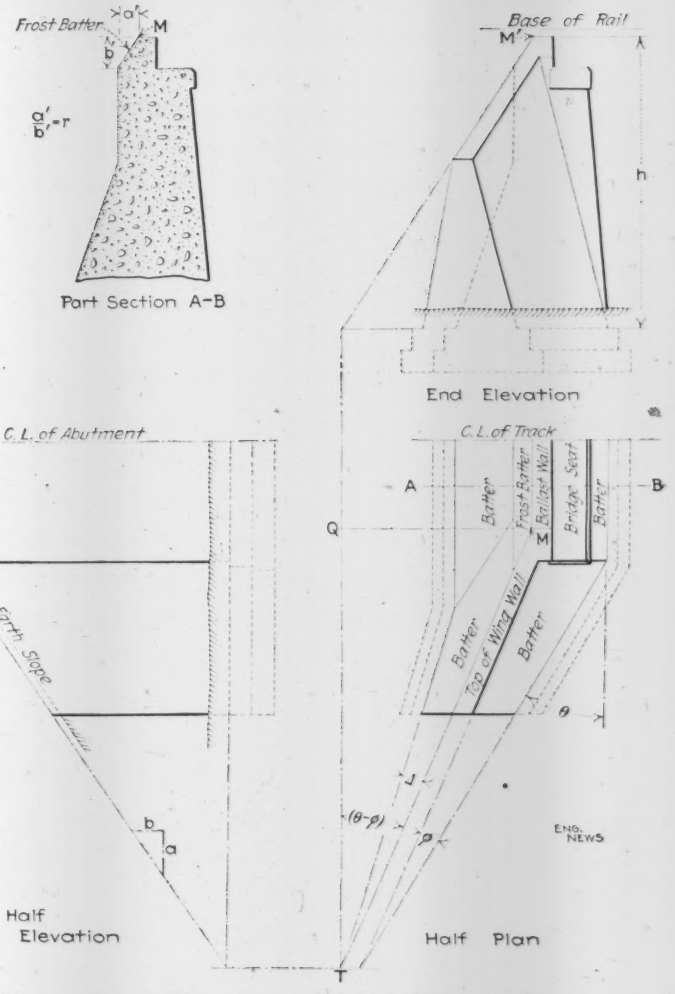

FIG. 2. SKETCH OF WING ABUTMENT FOR THROUGH SPAN; ANGLE OF WING COMPUTED

In the particular case shown by Fig. 1, $B'D'$ passes through P, a known distance from the top of the ballast wall and on line with its face. Under these conditions the distance $C'D'$, or CD, is obtained from the similar triangles $B'KP$ and $C'D'P$, in which $B'K$, KP, and $C'P$ are known. Referring to the plan, in the triangle BDE we have the sides BD and BE, and the included angle DBE. We can readily find ϕ, as well as the other elements of the triangle.

To find the batter on the front of the wing-wall proceed as follows:

Assume the wing-wall drawn to its full length to the vanishing point, then cut it at any height by a vertical plane parallel to the center-line of track.

Let h' = Height of wing where it is cut,

s = Slope of wing in a vertical plane perpendicular to the center line of track,

b = Rate of batter on front of wing per unit of height,

Draw FG and FL perpendicular to BE and EE', respectively.

Then,

$$FL = sh'; \quad EF = \frac{sh'}{\cos(\theta - \phi)}$$

$$h'b = FG = EF \sin\phi = \frac{s h' \sin\phi}{\cos(\theta - \phi)}$$

Dividing by h' we have

$$b = \frac{s \sin\phi}{\cos(\theta - \phi)}$$

After deciding the width of base of the wing-wall at its junction with the breast, a reasonable batter is given to the back of the wing, and generally the direction of the toe of the back batter ($E'G'$) is within the reasonable limit.

If b' is the rate of batter per unit of height on the back of the wing-wall, we have the triangle $E'F'G'$,

$$Side\ E'F' = EF = \frac{sh'}{\cos(\theta - \phi)}$$

and

$$h'b' = F'G'$$

$$\sin\Delta = \frac{F'G'}{E'F'}$$

$$= \frac{h'b'\cos(\theta-\phi)}{sh'} = \frac{b'\cos(\theta-\phi)}{s}$$

which is identical with Mr. Jensen's formula for Δ.

WING SLOPE FIXED BY FROST BATTER

In another design of abutment, shown by Fig. 2, the distance corresponding to BD of Fig. 1 is not directly determined. But from the nature of the design the point M is known, which is the end of the ballast wall at its back. It will be noticed that in this design the slope of the wing-wall, projected on the vertical plane through the center-line of track, coincides with the slope of the frost batter. In other words the battered surface immediately behind the ballast wall would form, if produced, the back line of the top of the wing-wall in the end view, as can be seen by a comparison of the end view with the partial section of the breast.

Given h, the height from the top of the ballast wall to the top of the footing; r, the rate of frost batter per unit of height; and s, the slope of the wing-wall on a vertical plane perpendicular to the center-line of track; we have in Fig. 2:

$$TQ = sh; \quad MQ = rh;$$

$$\tan MTQ = \frac{rh}{sh} = \frac{r}{s}$$

or angle $MTQ = \tan^{-1}\frac{r}{s}$.

Hence $\phi = \theta - \tan^{-1}\frac{r}{s}$.

These two cases, to which a number of others can easily be added, go to show clearly that the designer must utilize his knowledge in trigonometry and descriptive geometry in each special case.

Underdraining a Low-lying Athletic Field

By W. Waldo*

In planning the athletic field for the new Rice Institute, Houston, Tex., the engineer was confronted with the same general conditions which surrounded his work on the Institute grounds as a whole. The site had been selected arbitrarily for convenience of access and economy of space rather than on account of its natural fitness for an athletic field from a constructive standpoint.

The field is located on some of the lowest ground on the property of the Institute and the site was almost divided by an open subsidiary drain running to the creek, which is shown on the accompanying plan. It was necessary to fill this drain and raise the whole area to an elevation above the flood water level of the creek.

The drawing shows the plan of subsurface drainage adopted better than it can be described, and the tests to which the system has been subjected have determined the success of the plan.

Our climate is one of extremes so far as wet and dry weather is concerned, and the drains are arranged so that in extremely dry weather the field can be irrigated through them. This is accomplished by closing the drains where they enter the concentration manhole and turning on the water cock in the catch basins.

The running track is filled with 8 in. of crushed coke cinders thoroughly rammed, not rolled. The concourse between the track and the bleachers is laid over with 5 in. of cemented shell, which is the natural clam shell dredged from Galveston Bay, and contains a certain percentage of blue clay.

Outside of the bleachers, and completely surrounding the field is a gravel roadway and parking space, and the whole area is protected on the high sides by protection ditches to intercept surface water.

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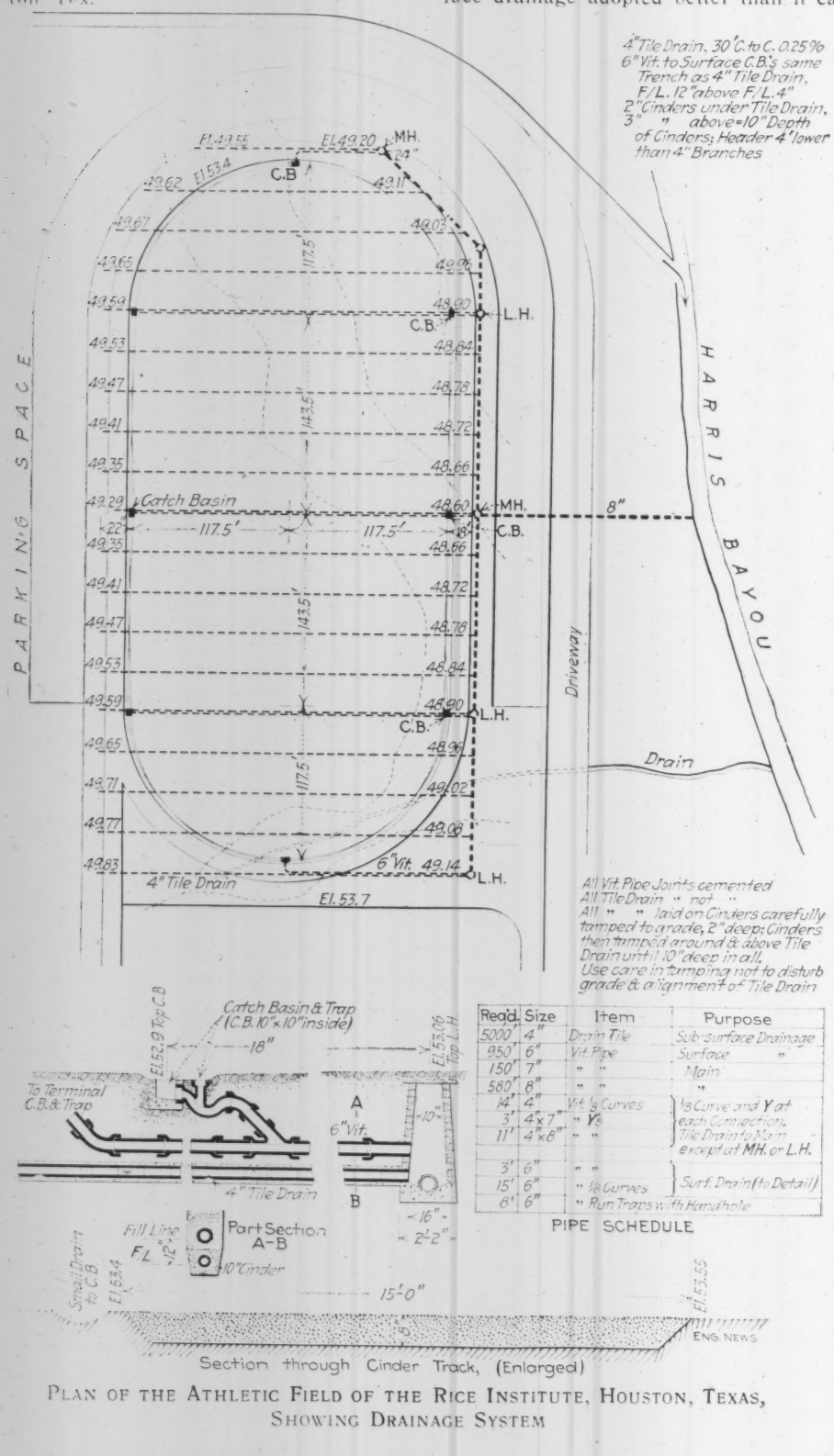

4" Tile Drain, 30' C. to C. 0.25%
6" Vit. to Surface C.B.'s same Trench as 4" Tile Drain, F/L. 12" above F/L. 4"
2" Cinders under Tile Drain, 3" " above=10" Depth of Cinders; Header 4" lower than 4" Branches
PARKING SPACE
HARRIS BAYOU
Driveway
Drain
Catch Basin
C.B.
L.H.
MH.
8"
6" Vit.
4" Tile Drain
El. 53.7
117.5'
143.5'
All Vit. Pipe Joints cemented
All Tile Drain " not "
All " " laid on Cinders carefully tamped to grade, 2" deep; Cinders then tamped around & above Tile Drain until 10" deep in all.
Use care in tamping not to disturb grade & alignment of Tile Drain

Read.	Size	Item	Purpose
5000'	4"	Drain Tile	Sub-surface Drainage
950'	6"	Vit. Pipe	Surface "
150'	7"	" "	Main "
580'	8"	" "	"
14'	4"	Vit. ⅓ Curves	⅓ Curve and Y at each Connection, Tile Drain to Main except at MH. or L.H.
3'	4"×7"	" Ys	
11'	4"×8"	" "	
3'	6"	" "	Surf. Drain (to Detail)
15'	6"	" ⅛ Curves	
8'	6"	" Run Traps with Handhole	

PIPE SCHEDULE

PLAN OF THE ATHLETIC FIELD OF THE RICE INSTITUTE, HOUSTON, TEXAS, SHOWING DRAINAGE SYSTEM

The Survey of the Tropical Island of Pemba*

By J. E. E. Craster†

Pemba is a coral island some 400 square miles in area, lying 60 miles north of Zanzibar, and forming part of the Zanzibar Protectorate. Upon it is grown more than half the world's total supply of cloves. The clove plantations, which cover about a quarter of the island, are owned by Arabs, who until the abolition of slavery in 1897 employed nothing but slave labor. It is said that in the flourishing days of the slave trade 14,000 slaves a year were imported into Zanzibar and Pemba, of whom the majority died within a year or two.

The temperature of the island varies but little throughout the year. The minimum cold weather temperature at night is about 74°, and the maximum day temperature in the hot weather 90°. Heavy rain falls throughout April and May, and there is a second rainy season in November. Showers are of almost daily occurrence during the other months of the year, and the air is always saturated with moisture.

The island is about 40 miles long, and 6 to 12 miles wide. Some 2 or 3 miles off the western coast lies a chain of small islands, and on the eastern shore of the largest of these a base was selected. Nearly the whole length of the base lay below high-water mark, and the central portion below the low-water mark of neap tides, so that the measurements could only be made at low water during spring tides. This involved a delay of

*Abstract of an article in the "Professional Memoirs," Corps of Engineers, U. S. A., No. 18, Vol. IV.

†Captain, Royal Engineers, British Army.

ten days between the first and second measurements. The measuring party were often working in water a foot or more in depth, but this caused little difficulty and had the advantage of keeping the tape at a uniform temperature.

The base was measured with a 300-ft. steel tape. The mean of two measurements gave a length of 13,817.287 ft., with a probable error of 0.4 in., or 1 in. 373,000 of the total length.

An astronomical azimuth was observed with a 5-in. micrometer microscope theodolite at Weti, a small town on the western coast of Pemba, from which both ends of the base were visible. Two pairs of east and west stars were observed, and the value found had a probable error of 0.5″.

The latitude of Weti was found from astronomical observations taken with a 5-in. theodolite. Four pairs of north and south stars were used. The latitude was found to be 5° 3′ 48.3″, with a probable error of 2.9″.

Trigonometrical triangulation was only possible between the chain of islands and the west coast of the main island. Very few points could be fixed inland, owing to the lack of any prominent hills, and the dense vegetation. A large amount of clearing was required at each trigonometrical station, and on the average it took one day to establish each.

Some of the trigonometrical stations were established in mangrove swamps and below high-water mark, no other site being possible. The atmosphere was very thick, and great difficulty was found in seeing the trigonometrical beacons, especially when they were standing before a background of waving palms. Every form of beacon was tried, and the only one that proved to be visible was a straight pillar, a foot in diameter, and 10 to 15 ft. in height. These pillars were made of light wooden poles padded with grass or brushwood to the required thickness. If the background was dark, the beacon was covered with white cotton cloth, but if the background was sea or sky, the beacon was left uncovered. A large flag was placed on the top of each beacon.

Many beacons were destroyed by the natives as soon as they were erected, and the materials stolen. Native police were told off to protect the beacons, but no arrests were made. Eventually, it was found that the best way of protecting the beacons was to slash the white cloth with a knife so as to render it useless for wearing purposes. An even more effective way would probably have been to splash the white cloth with red paint, which the natives would certainly have taken for blood. They regard a blood-stained cloth as "very bad medicine."

At three points tree stations were constructed. A large tree was cut some 20 ft. from the ground, and the top of the stem then formed an observation pillar on which the theodolite stood. The legs of the instrument were not used. Round the stem, and some 4 ft. below the top, a platform of sticks lashed with creepers was built for the observer.

It was found necessary to cut away all the branches from round the stem, otherwise the tree shook in the slightest wind. If a tree station more than 20 ft. high is required, a very large tree must be selected, or the stem will sway a little to every movement of the observer.

When the triangulation had been carried as far as possible, topography of the western coast was begun. Theodolite traverses were run across the island from trigonometrical stations on the western coast. The traverses involved very heavy clearing, and as the clove harvest was just beginning, the labor was very limited. Consequently, progress was very slow. At first the traverse lines ran through the clove plantations, and here great care had to be exercised to select the lines that involved least cutting, as heavy compensation had to be paid for each tree cut. After the clove plantation had been left behind, the lines entered thick bush with only occasional patches of cultivation. In the thicker parts of the bush, felling the trees was not sufficient, as they lay piled up to a height of 6 or 8 ft. in the traverse clearing. It was, therefore, necessary to cut up each tree and remove it piecemeal. This reduced the rate of progress to about 900 ft. a day.

Over the trees grew in great luxuriance a creeper bearing a fruit like bunches of white grapes. The leaves of this creeper when crushed under foot gave out a strong aromatic scent that tainted the atmosphere in the whole neighborhood, affecting even the native boys with a feeling of suffocation and giddiness. Another creeper bore pods covered with minute "hairs," causing the pods to appear as if made of yellow plush. When the creeper was shaken, the air was filled with these tiny yellow "hairs," which penetrated the pores of the skin and caused intense itching and irritation.

On several occasions the traverse line had to be altered, to avoid disturbing the nests of wild bees. A professional honey gatherer accompanied the cutting party, to burn out and remove the nests, but when there were many nests close together even the professional would not tackle them.

It was not found possible to close the traverses on each other, as had been the original intention, owing to the heavy clearing that would have been required. The native boys also refused to do any more work in the dense bush belt that fringes the eastern coast of the island.

Six theodolite traverses were run across the island. Their total length was about 40 miles, and there were 218 traverse stations. The labor involved in calculating the coördinates and heights of the latter was very heavy.

Owing to the absence of prominent hills and the density of the vegetation, ordinary plane table methods could not be used in taking topography, and the following procedure was, therefore, adopted. Prismatic compass traverses were run between trigonometrical and traverse stations, so as to cut the country up into triangles of 3 to 5 miles side. These traverses were plotted on foolscap paper on a scale of 6 in. to 1 mile. The closing error of each traverse was distributed among the traverse stations, and the corrected coördinates of each station measured. The traverse stations were then plotted on the plane tables from their corrected coördinates, and the detail was plotted from the traverse field books.

The topographers subsequently took the plane tables into the field and filled in all the remaining detail, and the contours, by plane table traverses run between the points already fixed. As these traverses were short, the closing errors were small, and could be adjusted in the field. The plane table was set by means of the trough compass.

As it was generally impossible to see more than 30 ft. in any direction, a native boy was sent about 600 ft. ahead to act as a point, and a compass bearing was taken—or, in the case of a plane table traverse, a ray drawn—on the sound of his voice. At 600 ft. the probable error in the bearing was found to be about 2°. The error, however, increased very rapidly with the distance, and at 1200 ft. it was about 5°.

Great difficulty was found in locating the point, if high ground intervened between him and the observer. The point was, therefore, instructed to halt at the top of each hill. It was found that a low pitched note was much more easily located than a high pitched one, and for this reason a boy with a clear bass voice was always selected as a point.

Contouring was very difficult, as no observations of any value can be taken with a clinometer on the sound of a voice. Fortunately, the valley bottoms, though often swampy and impassable, were fairly clear and their slopes very uniform, so that it was possible to determine their general gradient by a few clinometer observations. The topographers, therefore, mapped the main valleys as soon as possible and marked along them the heights as calculated from their general gradient, starting from the high-tide mark. The heights of the hills were afterwards fixed by reference to points in the valleys. The determination of heights by aneroid readings were tried, but proved a complete failure owing to the atmospheric pressure being very variable.

The country offered many difficulties to the topographers. Steep razor-backed ridges, about 200 ft. in height, alternated with deep crooked valleys. The bottoms of the valleys were marshy and often impassable, and tidal creeks filled with im-

penetrable mangrove swamps intersected the country, so that long detours were necessary. Owing to the nature of the work the topographers could only survey half a square mile a day. Much time was also lost from wet weather.

Great care was necessary to obtain the correct names of the various districts and villages, as the common pronunciation often gave no clue to the spelling. Many of the villages had opprobrious names, such as "The Fools," "Fly Blown," etc., which the inhabitants would not admit, but which were used with gusto by their neighbors. One little congregation of poverty-stricken huts was endowed with a name which meant "Because I have nothing, my brothers won't visit me." Lists of all names that would appear on the map were made out and forwarded to the assistant collector of the district who, after consultation with the headmen, wrote the correct form against each name.

Each officer and noncommissioned officer was equipped with a double fly ridge tent, camp furniture, cooking utensils, and a filter, so that he could camp alone if necessary. In addition, there were two mosquito-proof houses; one for the officers and one for the noncommissioned officers. The houses consisted of a mosquito curtain 8x8x8 ft. in dimensions. A small sheet, 12x12x8 ft., thatched with palm leaves, was built at each camping ground, and the mosquito net hung from the rafters. The windward sides were filled in with palm leaves to keep out the driving rain, but the leeward sides were left open. Without these mosquito-proof houses it would have been impossible to do any work after sunset, and there is no doubt that they saved the party from a great deal of fever.

Encouraged by the example of the survey party, one of the Zanzibar Government officials employed by the agricultural department to supervise the gathering of the clove crop camped in the interior of the island for some months, but did not use a mosquito-proof house. He suffered from repeated attacks of fever, and died in Zanzibar a few days after leaving Pemba. With the exception of the survey party, no other European has ever attempted to camp for more than a few nights in the interior.

Owing to the very heavy dews and frequent rain, the native boys and servants could not bivouac as they do in most parts of Africa. Native huts were, therefore, hired for them, and when these were not available thatched huts were built.

Five grains of quinine a day were taken by every member of the party as a preventive, but all suffered from malaria. There were no cases of blackwater fever, though it is very prevalent in the island, and often follows an attack of malaria if the patient returns to work too soon. The doctor, therefore, gave instructions that no malaria patient should return to work until four or five days after the fever had left him.

Two of the party suffered from severe congestion of the liver, accompanied by high fever, as the results of chills. It has been suggested by a doctor of considerable African experience that these attacks may have been due to the bite of a tick, which produces similar symptoms in animals. This is a disease that only a doctor can diagnose, and if the proper remedies are not applied early abscess of the liver is likely to follow.

Owing to the necessity for finishing the survey before the rainy season, it was necessary to work ten hours a day for seven days a week, and at the end the whole party was much exhausted by the hard work and the effects of malaria. In fact, ten months may be taken as the maximum time that a European surveyor can work at a stretch in the unhealthier parts of Africa. At the end of that time he should have at least two months' rest in a bracing climate before he returns to work.

Notes

Tests of the Condition of Creosoted Timbers, which have been in use for 29 years as stringers supporting ties of the long bridge of the New Orleans & Northeastern R.R., are being made in the laboratory of experimental engineering at Tulane University, New Orleans. The specimens are of 6x16-in. cross-section and supports for cross bending are 12 ft. apart.

Leveling with a Rifle—The demand for "quick" methods in field engineering brings out a radical innovation every little while. The following newspaper account records one of them:

> "Zan" Ervin, who has the contract for putting in the abutments to the new bridge on South Market St., has a new way of finding a level. He had one abutment up and in order to ascertain the height to build the other to be on a level, set up a board on the opposite side of the stream "took down" his rifle, laid it across the top of the abutment and fired, the bullet going true and striking the board in exactly the right spot for the top and level of the abutment.

D. McGillivray, McArthur, Ohio, informs us that the only thing not true in the item is the statement that the contractor set up a board to fire at; he fired into the clay bank. If leveling with the rifle should become very popular it might cut down the number of rodmen. But a good way of working the collimation adjustment on the new style of level would have to be found.

Sinking a Mine Shaft by grab-bucket excavation for the Todd & Stambaugh Co., at Hibbing, Minn., developed some interesting features. The excavation was done inside a concrete shell 29 ft. diameter on the outside and 21 ft. inside. At the center was a 12-ft. temporary shell or tube, this being filled with sand for the purpose of additional weight, to enable the tube to penetrate the clay and other material at the bottom. Difficulty was experienced in keeping the shaft vertical, and at a depth of about 140 ft. it was 21 in. out of plumb. It had been 36 in. out a short time before, but the contractors remedied this by means of an eccentric loading of 500 tons on the concrete shell. At two different periods a flow of water was encountered. Besides the grab buckets, a special stone grapple was used for handling boulders and stone. In one case, the bucket (which weighed approximately 3750 lb.) brought up a stone weighing about 6½ tons. The shaft sinking was done by the Foundation Co., of New York.

The Preparation of Contract Plans by the Board of Water Supply of the City of New York was described by C. F. Bell in a paper read at a meeting of the Municipal Engineers of the City of New York as follows:

The steps of the system under which the Designing and Drafting divisions work together are: preliminary studies on detail paper are made, approved by the Designing and Department Engineers and sent to the Assistant Engineer in Charge of Drafting with an instruction sheet. A drafting "progress form" is then made out, on which the contract number, sheet number and final accession number, the accession numbers of the original studies, a brief title and the date when the work is received are filled out. This form is then inserted in the "progress book," and the studies, instruction sheet and a ticket marked "ready to trace" are all pinned together and placed in a case until a draftsman is available.

When the work is assigned to the draftsman, his name and the date are written on the progress form, an entry is made on the monthly record which shows work begun and completed and the job is turned over to him with any additional instructions that may be necessary.

Green is a danger sign with us, so anything either underlined or ringed with green is omitted on the final tracing although it may go on later and is used in plotting and for checking.

When everything has been traced, the draftsman's initials and dates are placed on the "ready to trace" ticket and the work is turned in. A corresponding entry is made on the progress form and another entry and a "drafting check" ticket are made for the checking, all steps being entered and ticketed this way. This check is for drafting errors only, not those of design; errors are indicated in blue pencil on the tracing.

When this check has been accepted and the corrections made by the man who did the tracing, a blue print is sent to the man in charge of the design who has the print checked in red, and corrections and additions indicated in yellow. The draftsman then makes corrections from this print, a check is given and the printer then prints the title and sub-titles, after which the tracing is sent for final check and signatures.

A complete new check is then given and after any final changes have been made, the drawing is signed. If it is to be used for study purposes no signatures are necessary, except on drawings for issue outside of the Board's offices. On working drawings, the Designing, Department and Chief Engineers sign, while on Contract drawings only the Designing and Department Engineers sign, as the Chief Engineer signs a title page for all the drawings, except for small contracts needing only four or five sheets in which case a title page is not used and the Chief signs every drawing. One or more of the Consulting Engineers also signs the title page or all sheets for a contract drawing the same as the Chief Engineer. The customary proceedings regarding the signature of contract drawings, were in-

vestigated by inquiry among the government and state departments, the Pennsylvania R.R. and others; the present system was developed from these.

Magnifying Power of Telescopes—A simple method of finding the magnifying power of any form of refracting telescope, except the Galilean, is to find the ratio of the true aperture to the diameter of the emergent pencil of rays at the eye-piece. This method is familiar to nearly every surveyor, and an accurate means of using this method will be found explained in detail in Engineering News of Mar. 7, 1901, p. 169. If the telescope is of the Galilean type the emergent rays are divergent and this method will not answer. To determine whether or not a field-glass or telescope is of the Galilean type, examine the eyepiece, and if the telescope is of the Galilean type one or the other face of the eyepiece (always simple) will be concave. In this case the following simple method which is applicable to any type of telescope, described by Lieut. Wm. F. Endress in the "Professional Memoirs" of the Corps of Engineers, U. S. A., may be used.

Measure the angle subtended by some clearly defined distant object by means of a surveyor's transit. The angle should preferably be between 2° and 4°. Call this angle A. Now take the telescope or field-glass whose magnifying power is desired and focus it on the same object. Reverse it and place it in front of the object glass of the transit, its object glass facing that of the transit. Again measure the angle subtended by the distant object, but this time as seen through the reversed telescope or field glass. Call the angle so obtained a. The magnifying power $= \frac{\tan \frac{1}{2} A}{\tan \frac{1}{2} a}$.

If no transit is available it is necessary to determine the magnifying power of a Galilean telescope by analysis of the lenses. In all cases, the magnifying power $= \frac{\text{focal length of object glass}}{\text{focal length of eyepiece}}$.

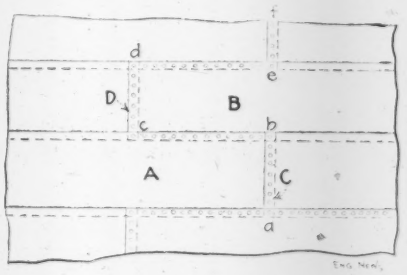
The accompanying table gives the results obtained by the different methods mentioned and their comparison with the certified magnification by the U. S. Bureau of Standards.

Eliminating Muskrat Troubles in a Gravel Road Fill—At the meeting of the American Road Builders' Association in Cincinnati, Ohio, Dec. 5, R. H. Dixon, Jr., County Roads Engineer, Cambridge, Md., propounded an interesting road maintenance problem. He said he had a seven-mile gravel road across a marsh, on a fill of about 1½ ft. above the water, over which he could walk one day and find it in excellent condition and on the next day find a dozen or more bad holes caused by cave-ins over muskrat burrows. He appealed for advice on a remedy.

George W. Cooley, State Engineer of Minnesota, rose to the occasion and said that he had once met with the same problem on a similarly situated road. He had first recommended a small boy and a shot-gun, but this had not proved effective. Later in traveling over an old corduroy road through a swamp he found that every hollow log contained a muskrat's nest; and an idea then came to him. Going back to the gravel road over the marsh he inserted a blind drain every 50 or 100 ft., the inner ends being blocked with flat stones. The muskrats immediately occupied these blind drains and caused no more trouble by boring holes under the roadbed.

A New Point in Tank Design—A novel argument is advanced by J. T. Maguire, Washington, D. C. In designing tanks, standpipes, etc., the present universal practice is to design each ring of plates as if it stood by itself; the longitudinal joint, the weakest point of the ring, is designed such that each lineal foot of joint can carry the pressure on a 1-ft. height of shell within the allowed stress intensity. But, the vertical joints of successive rings being staggered, Mr. Maguire claims that the joint of one ring is in part spliced by the unbroken metal of the next ring; he claims that on this account the rings should be designed on the basis of the average between strength of joint and strength of plate. Mr. Maguire writes us as follows:

An examination of the common formulas for tank design indicates that each ring of the tank is designed to resist the fluid pressure within, by itself, on the basis of no assistance whatever



from the adjacent rings above and below. This would seem to presume that the weak point in one ring of the tank was immediately over the weak point of the ring below, or in other words that the joints were continuous from top to bottom of tank. This is, of course, not the case.

Taking plates A and B, in the sketch herewith, as two adjacent plates in a cylindrical tank, we see, by the ordinary methods of analysis, that failure can occur in only two ways: (a) By rupture along line abef, or (b) by rupture along the broken line abcdef. In an ordinary tank the value of the rivets along bc and de, in single shear, is sufficient, with the value of the joint D, to equal the value of the plate B along line be. Therefore, the tank would in most instances fail along line abef.

The proper method of analysis, then, would seem to be: First determine the value of joint D plus rivet-lines bc and de; if this is equal to or greater than the value of plate B, then the value of plate B would control the design for this strake, joint C controlling the design for the lower strake. Considering the tank as a whole, then, it would be perfectly proper to allow a strength of 100% for each alternate plate, and a strength equal to the efficiency of the joints for the remaining plates.

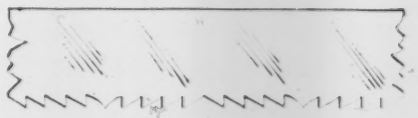
Assuming a joint efficiency of 70% for the vertical joints, the writer believes that the design of the tank could be based on a joint-efficiency of ½ (100 + 70) = 85%, for all strakes. This, as will readily be seen, would mean a saving of about 17% in the weight of the side walls without increasing the unit-stress beyond the presumed stresses in tanks designed in consonance with the requirements of the standard text-books.

The above argument amounts to the claim that initial failure of a tank cannot occur in one ring alone, but that two consecutive rings would have to fail simultaneously—a claim that can hardly be given serious consideration. On the same basis it would be possible to cut out the rivets in the vertical joints of alternate rings, substitute a little solder to prevent leakage, and then have the tank resist half the full head of water!

Even apart from the phenomena of actual rupture, however, it is clear that the stress in each vertical joint is not reduced by the presence of the full metal of the adjoining ring, since the circumferential elongation per unit of circumference is the same at the joint as in the unbroken metal.

Prizes for Welding-Test—A new test for welds is wanted. Prizes are offered by the "Zentralbüro für Acetylen und autogene Metallbearbeitung," of Nürnberg, Germany (Gugelstrasse, 54), for a method of detecting slag inclusions, seams, and overheating in autogenous welds, without injuring the weld. The test appliances should preferably be simple and portable. In case complete solutions of the problem are not obtained partial solutions (such as will serve in the majority of cases) may be awarded the prizes. Two prizes are offered, 1000 M. ($240) and 500 M. ($120). Solutions should be sent in writing, without name, but marked with an identifying word or phrase, the latter being also stated with the author's name in a separate sealed envelope. They will be received until July 1, 1913. A jury of six, five prominent technical men and professors and a representative of the company, will judge the competition.

A Double-acting Hacksaw—Hacksaw blades made with alternate groups of teeth facing forward and back are made by A. Reitlinger, 201 William St., New York City, as a new article. The sketch shows the construction fully; the teeth



REITLINGER DOUBLE-ACTING HACKSAW

are set, as usual, alternating, the triangular tooth between oppositely facing groups being set like the others.

This saw is said to cut faster and cleaner than the ordinary hacksaws, and to cut iron pipe without breaking teeth.

Sag in Steel Tapes on Slopes—A correspondent in Fresno, Calif., writes: Can some of your readers tell me whether there would be an appreciable error if formulas for sag, etc., which are derived for a tape held with both ends at the same elevation (or a slight difference of elevation), are applied on steep slopes of from 30 to 45°? Perhaps some of our readers have investigated this point, and if so, we should be pleased to hear from them.

TABLE SHOWING ACCURACY OF DIFFERENT METHODS OF DETERMINING THE MAGNIFYING POWER OF TELESCOPES

Glass	Maker's Rating	Bureau of Standards	Lens Analysis	Angles by Transit	Measurements of Apertures	Eye Comparison
Lemaire No. 1	....	5.1	4.8	4.8	....	4.6
Lemaire No. 2	....	....	4.5	4.5	....	4.6
Bardou	....	6.6	6.4	6.4	....	6.3
K. & E. No. 6931	8	6.8	6.9	6.7	....	6.6
K. & E. No. 6932	9	7.5	7.5	7.4	....	7.0
Signal Corps, Day	6.5	6.0	6.0	5.9	....	5.7
Signal Corps, Night	4.5	3.8	3.9	3.7	....	3.6
Busch	10.0	10.1	...	9.9	10.3	10.0
B. & L.	10.0	10.5	...	10.5	10.5	10.5

Designing Diagram for Cantilever Reinforced-Concrete Retaining-Walls

By W. D. Hudson*

The writer has found the accompanying diagram convenient in designing L-shaped reinforced-concrete retaining-walls. The purpose of the diagram is to ascertain quickly the proper thickness and reinforcement under different conditions of loading of the *vertical slab* in walls of the type shown in section on the cut. Its use may best be explained by means of examples traced out on the accompanying diagram.

Example 1—To design the vertical slab at depth 15 ft., for a wall with surcharge angle of 30° and angle of repose 30°; i.e., $D = 15$, $\phi = 30°$, $\theta = 30°$.

At depth 15 on the left-hand ordinate, proceed horizontally to the heavy dashed line *A-A*, and read on the nearest "thickness curve," 20 in. Proceed thence vertically upward and read at top of diagram 1.84 sq.in. per lin.ft. for the amount of reinforcing.

In case any other thickness is required by other conditions, the reinforcing area is read in the same manner; thus, if 26-in. thickness is required, reinforcing area equals 1.4 sq.in. per lin.ft.

Example 2—To design a slab at depth 15 ft. for a wall with level surcharge and 30° friction angle; i.e., $D = 15$, $\phi = 0$, $\theta = 30°$.

Where depth 15 intersects curve marked "$\theta = 30°$, $\phi = 0$," proceed vertically to curve marked "Reference Line," thence across to dashed line *A-A*, and read slab thickness 13½ in. Vertically above read reinforcement of 1.24 sq.in. per lin.ft.

The thicknesses of slab given are net thicknesses. Stresses used are: tension in steel, 16,000 lb. per sq.in.; compression in concrete 650 lb. per sq.in.; straight-line formula used in the analysis. As may be seen, the slab may be designed for five different conditions of loading.

It should be noted that the size of the base slab must be determined by some other means than given by this diagram.

For Any Slab—The diagram can be used for *any slab* of known bending-moment, as follows: Locate the bending-moment on the scale at the foot of the diagram, follow upward to "reference line," and thence follow horizontally to the appropriate slab curve. Of course, where the horizontal meets the dashed line, the most economical thickness and reinforcement are found, i.e., the design which realizes the full allowed stresses in concrete and steel, respectively.

*Missouri Pacific Ry. Co., St. Louis, Mo.

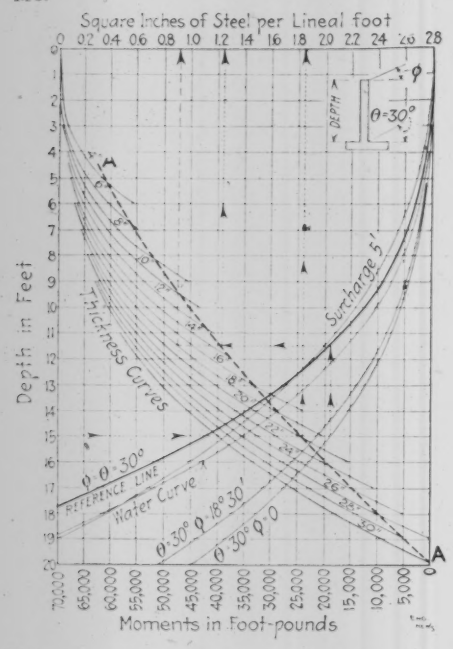


Diagram for the Design of the Vertical Slab in Cantilever Retaining Walls—W. D. Hudson

Diagram for Wire Spans

By O. von Voigtlander*

Problems relating to flat catenaries (stretched wire spans) involve many cumbersome calculations. Some of these may be handled by the use of tables published in electrical engineers' pocket-books, but even with such aids a lot of trial-and-error work remains. Consequently much time is apt to be lost before the best conditions of span length, tension, sag, etc., are determined. A new diagram, prepared by the writer to facilitate the calculations, enables the user to obtain quickly the correlative values of the several quantities involved.

The diagram, reproduced herewith, is of the parallel-scale type, and represents in effect a slide-rule designed for the particular work in question. The equations on which it is based are the same as recommended in engineers' handbooks. They are only approximate, but for all practical purposes give sufficiently accurate results. They should not, of course, be applied to spans having deep sag, as the error then becomes appreciable. The formulas are:

$$T = \frac{L^2 w}{8 d}$$

$$I = \frac{8 d^2}{3 L}$$

$$d = \frac{L^2 w}{8 T} = \sqrt{\frac{3 L I}{8}}$$

and

$$L' = L + I$$

in which the terms have the following meaning:

L = Length of span, feet;
L' = Actual length of cable, feet;
d = Sag at midspan, feet;
w = Weight of cable (or resultant of weight and wind), pounds per foot;
T = Tension in cable, pounds;
I = Difference between cable length and span, feet.

The value of w to be used when wind and ice are not considered is the weight of the wire (or wire and insulation). *When wind and ice are considered*, w is the resultant force, also in pounds per lineal foot. For finding the proper value of w rapidly, the subjoined table has been computed. [Note that the table gives the resultant force in pounds per 100-ft. length.]

Using the Diagram—Since the term L^2w occurs in several equations, its location on the axis marked L^2w in the diagram is to be found first: Draw a straight line through the value of span-length on axis marked L and the value of weight (or resultant force) per lineal foot on axis marked w; mark the point where this line intersects axis L^2w. This point is then used either to determine the deflection d for a specified tension T, or the tension in the cable for a specified deflection: Swing a straightedge around the point L^2w, and read off any pair of correlative values T and d.

When the deflection is known, a line can be laid through values of span-length L and deflection d, to intersect axis I at a value representing the difference between cable-length and span.

Other combinations of given terms may be used, and the unknown terms found by the diagram. The key sketch at the top of the diagram in all cases furnishes a guide to the correct use of the scales.

Temperature Changes—A change in length of the cable due to temperature change can also be found from the diagram. The formula is

$$l_t = l_o (1 + \alpha t)$$

in which

l_t = Length at a temperature t degrees Fahrenheit above the lowest assumed temperature;
l_o = Length at lowest assumed temperature;
α = Coefficient of expansion of the cable.

The diagram considers the quantity $l_o \alpha t$, or the increase of length. Since this quantity varies with t in a straight-line relation, the calculation for a single temperature in a given span will be sufficient, as all other temperature variations can be found from this one by simple proportion.

As it is customary to use a maximum temperature range of 150° F., this range was used in the construction of the diagram. Two scales are given, one for copper and one for aluminum. The figures marked on these scales represent span-length.

*303 Commonwealth Bldg., Jackson, Mich.

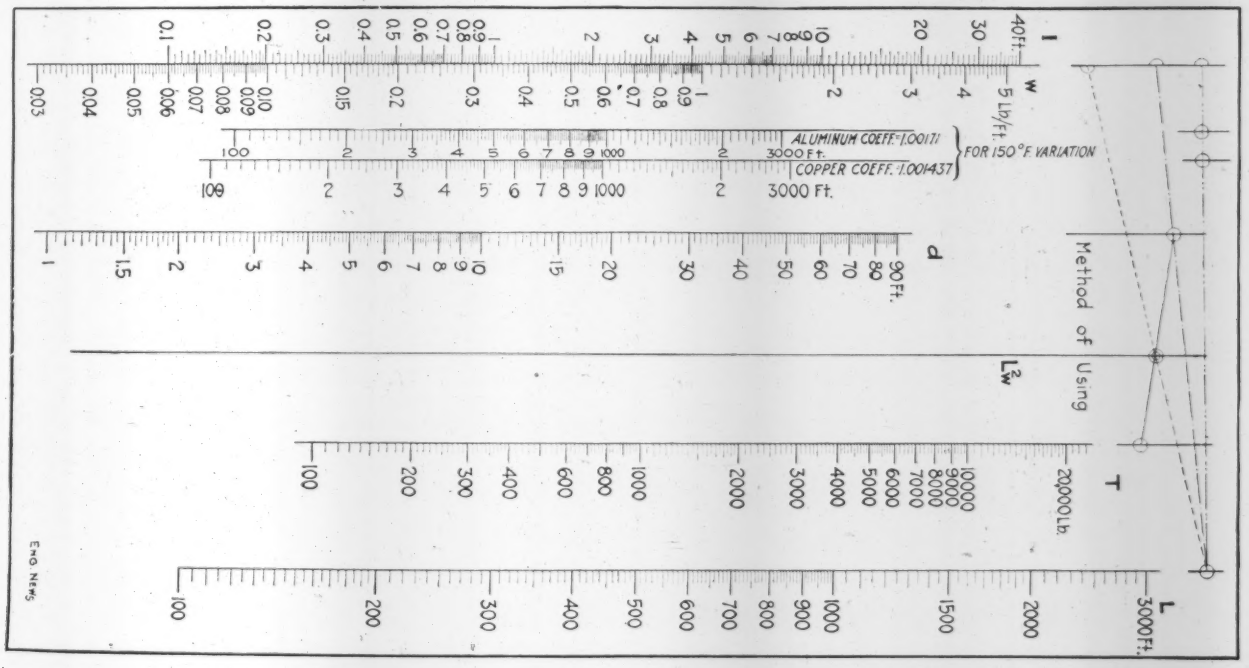


DIAGRAM FOR WIRE SPAN CALCULATIONS. O. VON VOIGTLANDER

To find the length of a copper wire 600 ft. long at normal temperature, after 90° F. increase of temperature: Draw a line through the point 600 on the L scale and the point 600 on the copper-wire scale, and read its intersection with the I scale. The result, 0.87 ft., is for 150° F. increase, and must be reduced to 90° by multiplying by $^{90}/_{150}$ or $^{3}/_{5}$, giving 0.52 ft. Then the length of the wire after expansion is 600.52 ft.

From this increase in length, 0.52 ft., we can find the increase in deflection by connecting I and L and reading the intersection on scale d.

EXAMPLE—Span 500 ft.; cable 000 copper; deflection under ice and wind to be not less than 4% or 20 ft. Find the tension, the length at 60° F. and the tension at 60° F.

In col. 15 of the table is found the value 171.34 lb. per 100 ft. or 1.71 lb. per ft. as the resultant of copper weight, ½-in. ice, and 65-mile wind. In the diagram, connect 1.71 on scale w with 500 on scale L, and locate the intersection with scale L^2W. Swing a straightedge about this point until it intersects scale d at 20 ft., and read value of 2680 lb. on scale T. This is the tension under ice conditions. The corresponding length of line is found by connecting 500 on scale L with 20 on scale d, giving $I = 2.13$, so that $L' = 502.13$ ft.

When the temperature increases to 32° F., or say by 30°, the weight of ice is removed and the wire is lengthened by increase of temperature. This latter increase in length is found by connecting 500 on scale L with 500 on the copper scale, reading the intersection on scale I (0.72 ft.), and reducing to 30° by multiplying by $^{30}/_{150}$; the result is 0.144 ft. The total length of wire now is 502.13 + 0.144 = 502.27, so that $I = 2.27$ ft. With this figure, and span 500, we find $d = 20.8$ ft. The weight w is equal to 0.657, from col. 9 of the table. Connecting 0.657 on scale w with 500 on scale L, locating the intersection on scale L^2w, and swinging a straightedge on this point so as to pass through 20.8 on scale d, the tension is read off as 990 lb. on scale T. The reduced tension will mean a slight shortening of the wire, which if desired can be computed and the corrected length of wire used in the diagram to get a closer approximation to deflection and tension; but this is usually unnecessary, as the correction is very small in all but exceptionally flat spans.

When the temperature rises to 60° F., the additional $I = {}^{28}/_{150} \times 0.72 = 0.134$ ft., or $L' = 502.274 + 0.134 = 502.408$, so that total $I = 2.41$ ft. For this latter figure, the deflection is found to be 21.75 ft., and using the previously-determined point on the L^2w scale the tension is found to be 950 lb., this being the tension at which the wire must be strung (if in 60° weather).

The results in this example as obtained from the diagram will be found sufficiently accurate for all practical uses.

TABLE OF WEIGHT, WIND-PRESSURE, AND RESULTANT FORCE, FOR WIRE SPANS

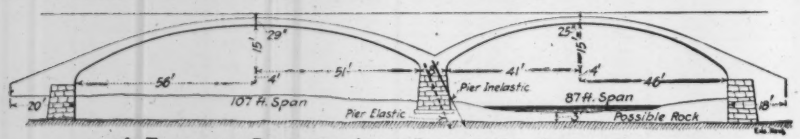
			Actual Wind Velocity = 65 mi. per hr.						Wind plus ½-in. Ice						Wind Vel. 80 mi. per hr. = 25 lb. per sq.ft. + ½-in. ice = 57.4 lb. per cu.ft.		Max. Allowable Stress on Section if Unit Stress = 25,000 lb. per sq.in.
			All Materials		Aluminum		Copper		All Materials		Aluminum		Copper		Copp. Rub. Cov. Double Braid		
Size B & S Gage	Diameter, in.	Area, sq.in.	Projection, sq.in. per lin.ft.	Horiz. Force, lb. per lin.ft.	Weight, lb. per 100 ft.	Resultant Force, lb. per 100 ft.	Weight, lb. per 100 ft.	Result. Force, lb. per 100 ft.	Projection, sq.in. per lin.ft.	Horiz. Force, lb. per lin.ft.	Vertical Force, lb. per lin.ft.	Result. Force, lb. per 100 ft.	Vertical Force, lb. per lin.ft.	Result. Force, lb. per 100 ft.	Result. Force, lb. per 100 ft.	Copper, Result. Force, lb. per 100 ft.	
4	0.234	0.0328	2.76	0.201	3.85	20.5	12.7	23.8	14.76	1.076	0.501	118.7	0.59	122.67	181.0		820
3	0.263	0.04134	3.12	0.227	4.85	23.2	16.0	27.8	15.18	1.108	0.533	122.9	0.65	128.54	187.2		1,034
2	0.295	0.0521	3.60	0.262	6.1	24.5	20.3	32.4	15.6	1.140	0.564	127.1	0.706	134.42	193.6	154.0	1,303
1	0.325	0.0657	3.96	0.289	7.7	29.9	25.5	41.1	15.96	1.166	0.604	131.4	0.782	140.6	200.0	160.0	1,643
0	0.378	0.0829	4.44	0.324	9.7	33.8	32.2	46.0	16.44	1.200	0.656	136.8	0.881	149.92	208.0	167.0	2,073
00	0.425	0.1045	5.04	0.367	12.24	38.7	40.6	54.8	17.04	1.244	0.713	143.2	0.997	159.36	219.5	178.0	2,612
000	0.475	0.13179	5.64	0.411	15.44	43.9	51.3	65.7	17.64	1.286	0.777	150.2	1.136	171.34	234.2	189.0	3,295
0000	0.524	0.1662	6.48	0.473	19.47	51.2	64.5	80.0	18.48	1.348	0.861	159.6	1.311	187.81	253.0	209.0	4,155
Kilo circ.m.																	
250	0.568	0.1964	6.96	0.508	23.0	55.8	76.2	91.6	18.96	1.385	0.944	166.9	1.476	202.29	267.0	228.0	4,910
300	0.637	0.2360	7.56	0.551	27.6	61.6	91.5	106.8	19.56	1.427	1.01	174.6	1.649	212.52	288.5	239.0	5,900
350	0.680	0.2749	8.16	0.595	32.2	67.7	106.8	122.3	20.16	1.47	1.092	184.2	1.838	234.85	308.5	252.0	6,873
400	0.735	0.31416	8.76	0.639	36.8	73.7	122.0	137.6	20.76	1.515	1.182	190.9	2.034	252.90	329.8	267.0	7,854
450	0.777	0.354	9.24	0.674	41.4	79.1	137.3	153.0	21.24	1.55	1.256	198.2	2.215	269.75	350.0	284.0	8,850
500	0.820	0.393	9.72	0.709	46.0	84.6	152.5	168.0	21.72	1.586	1.382	208.2	2.407	288.46	372.0	302.0	9,825
600	0.900	0.472	10.68	0.779	55.2	95.4	183.0	198.8	22.68	1.655	1.486	221.8	2.764	321.90	411.0	335.0	11,800
700	0.968	0.550	11.52	0.840	64.4	105.8	213.5	229.5	23.52	1.718	1.630	236.5	3.121	357.5	453.0	359.0	13,750
750	1.020	0.589	12.00	0.875	69.0	111.5	228.8	245.0	24.00	1.752	1.709	247.4	3.307	373.43	476.0	386.0	14,725
800	1.037	0.629	12.36	0.901	73.6	116.3	244.0	260.0	24.36	1.778	1.782	251.6	3.486	391.24	501.0	403.0	15,725
900	1.096	0.7075	13.08	0.954	82.8	126.3	274.5	290.0	25.08	1.83	1.914	261.8	3.821	424.5		437.0	17,688
1000	1.157	0.787	13.80	1.005	92.0	136.2	305.0	321.0	25.80	1.884	2.058	275.4	4.188	458.61		471.0	19,675
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

The sag at center should be about 2½% of span with wind but no ice.
The sag at center should be about 3½% to 4½% of span with wind and ½-in. covering of ice.

Letters to the Editor

A Dangerous Two-Span Arch Bridge

Sir—A concrete arch bridge is nearing completion in an Indiana city which is being constructed substantially according to the drawing herewith. It will furnish an interesting illustration of the effect of unbalanced thrusts upon a pier. Analyzed according to the elastic theory from springing to springing, but assuming the pier inelastic, the resultant falls approximately as shown by the full line. If this solution is correct the structure cannot stand. An analysis based on the assumption that the pier is elastic, as well as the arches, deflects the resultant to the approximate position shown by the dotted line, just on the verge of instability.



A Two-span Concrete Arch Bridge Now Being Built

(The center pier is exposed to large unbalanced thrust, as the two spans are of different lengths but have the same rise.)

The contractor who is erecting this dubious structure intends to strike centers shortly. The bridge should collapse when centers are struck regardless of whether or not the earth fill is first added.

The unbalanced thrust on this pier amounts to about 17,000 lb. per lin.ft. without earth filling. The earth fill is to be added after centers are struck, a particularly dangerous method in this case. The addition of the entire earth filling will increase the unbalanced thrust by a slight amount, but will also increase the load on the pier.

The method of letting was well conducted, except that the appropriation was limited to $10,000 for a structure properly worth $14,000. General plans and detailed specifications were prepared by engineers, and bids received upon detailed plans submitted by the bidders, with award to the lowest bidder. The plans on which the bridge is being erected are, therefore, the contractor's plans, and failure, if it occurs, will undoubtedly have to be made good by the contractor. The contractor is experienced in steel-bridge building, but not extensively so in concrete-bridge building, and seems to have taken a very long chance in this instance.

The new bridge replaces a wooden bridge, on stone pier and abutments in good condition, which the contractor evidently believes rest on rock. The pier, however, shows evidence of resting on grillage, which might in turn rest on rock or hardpan, although it would be an unwise expedient.

It is extremely doubtful, therefore, whether the pier and abutments are on foundations that will support the load. But assuming that they are on rock, will the pier act as an elastic body under such conditions to the extent of saving the structure? It seems to me an extremely doubtful case.

Daniel B. Luten.

Indianapolis, Ind., Nov. 18, 1912.

[Lateral deflection or incipient tipping of the pier would reduce the thrust of the longer span and increase that of the shorter span, besides generating a certain amount of righting moment in the concrete over the pier. These forces may be great enough to prevent the collapse of the bridge on striking centers. But at the same time unforeseen stresses of unknown amounts would be set up in the arch rings, and the safety of the bridge must be under grave suspicion.—Ed.]

Failure of a Poorly Designed Concrete Arch Bridge

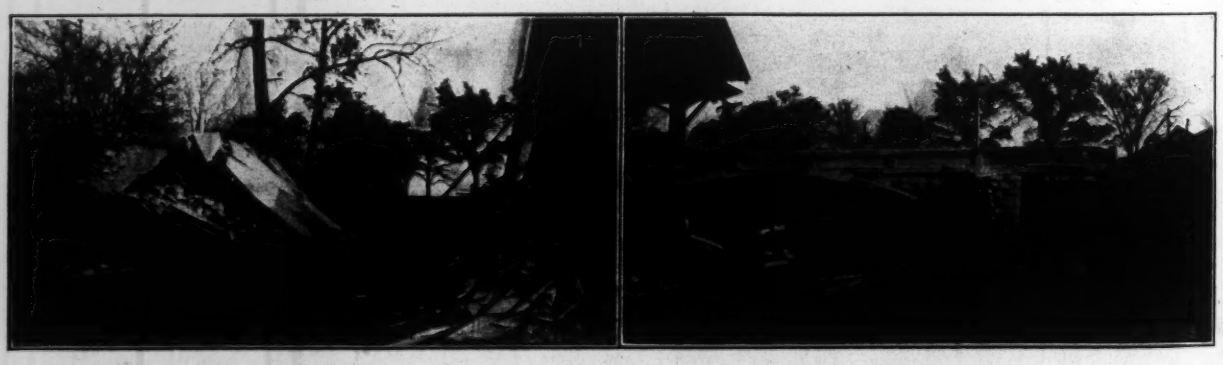
Sir—The photographs herewith illustrate the recent failure of a 38-ft. reinforced-concrete arch bridge on the grounds of the State Hospital for the Insane, Elgin, Ill. This can hardly be classed as a "concrete failure," as the unscientific design of the abutment is undoubtedly responsible for the collapse.

The bridge consists of two arches, one either side of the summer house at the left in Fig. 2. The structure was symmetrical around the summer house. Fig. 1 is a view of the left-hand arch which collapsed; Fig. 2 shows the other arch, which is still standing.

It will be noted in Fig. 2 that the right-hand abutment of the arch is a flimsy masonry pier, 2 ft. thick, and that this is separated from what should be the main abutment by an opening spanned by a beam. The concrete arches were put in to replace old wooden spans, and the masonry piers and abutments of the old bridges, built thirty or forty years ago, were used for the new arches. There seems to be no doubt that the left-hand arch failed by pushing over its outside pier, similar in construction to the one shown in Fig. 2.

R. L. Morrison,
State Highway Commission.

Springfield, Ill., Nov. 29, 1912.



Figs. 1 and 2. Two Arches Symmetrical about Summer House in Middle

(Collapsed arch on left was similar in design to one standing on right.)

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Engineering News

The Bridge Failure at Glen Loch, Penn.

The report on the Glen Loch wreck by the railway company's committee of officers presents the anomaly of a bridge-wreck investigation conducted without the participation of a bridge engineer. That the report makes no reference to the bridge failure beyond mention of a post-cap breakage, and that it does not comment on design, construction or maintenance of the bridge, is not, therefore, as surprising as it would be otherwise. Even the auxiliary fact that the cap-plate of Post 8 was placed in such position as to expose its weakest elements to the bending effect of the load is not emphasized by the committee.

Every bridge engineer will be deeply interested in studying the plan of the Glen Loch bridge, reproduced on another page of this issue. The unusual structural arrangement, made because of the sharp skew, will lead him to remark on the absence of lateral bracing. Of course, a 2° curve, with high-speed trains, means no insignificant amount of centrifugal thrust. Floor joists resting on stone abutment walls cannot afford much anchorage against this thrust unless they are held down to the abutment by bolts; but the shop drawings (appended to the committee report) show no such bolts, or holes for them.

A wrought-iron bridge over twenty years old will not be expected to show that conscientious strictness in detailing, and that well balanced provision for horizontal loads and secondary stresses, which are expected in present-day bridges. Even so, the post cap sketched in Fig. 3 may raise the question why the cap-plate detail with its heavy bending stress was accepted and continued in use for many years.

Altogether a structure built up like this one in several tiers of girders and beams, unstiffened against lateral motion, might be expected to show symptoms of its condition. In the mass of testimony taken by the committee we do indeed find some mention of what may be such symptoms, in the testimony of S. S. Scull, track foreman:

> A. Always had a good bit of trouble about receiving rough-track orders, especially in the winter time when the ground is frozen, complaint about bad track at that point. At the foot of the bridge would find some joints heaved up with the frost just when you come off the bridge.
>
> Q. Did you ever notice a tendency there for the track on the bridge to move to the north?
>
> A. Yes, on the east end of the bridge where there are two or three stones in what we call a back wall to the bridge, and the moving to the north had a tendency to move these two or three stones up against the main girder of the bridge and in turn that forced the bed-plate on which that main girder rests.
>
> Q. Ever heard of any loose rivets or bolts?
>
> A. Couple of loose bolts. I found loose bolts on the east end of the bridge. One that holds the plate into the abutment. They have never been taken out, they are there today. Bolts broken off down in the stonework.

The movement at the east end back-wall is mentioned in the committee's report; it is attributed to "wedging effect of the ballast," and is said to have "in no wise affected the integrity or safety of the structure." Yet it will appear to bridge engineers that it is a matter of fair question whether the girder bed-plate was "forced" by wedging action of the ballast or whether it may not be connected with the lateral non-rigidity of the bridge.

In passing, we may note also that the committee's report speaks of the breakage of the cap of Post 8 as though nothing else in the structure was injured. But it is certain that, if the northeast end of the central box-girder dropped 18 in., the adjacent panel-length of that girder must have been badly deformed; and bridge engineers would like to know just how it deformed. They would also inquire whether any other parts of the bridge were damaged. In short, they would regard such an investigation as being incomplete if it did not find, describe and analyze all effects of the train wreck on the bridge.

The Value of Guard Rails on Bridges

An interesting point noted in connection with the Glen Loch bridge wreck on the Pennsylvania R.R. bears on the behavior of track under derailed trains. Previous experience has many times demonstrated that both inner and outer guard-rails, but chiefly inner guard-rails, are a large factor in reducing the danger from derailments on bridges. In the Thoroughfare draw-bridge wreck at Atlantic City, in October, 1906, the wheels of a derailed train climbed the outer guard-rail and a car fell into the water, killing many. In all probability, inner guard-rails on that bridge would have held the train.

The Glen Loch wreck brings out the importance of close spacing of bridge ties and their secure anchoring against bunching to make the guard-rails most effective. The train at Glen Loch was derailed on the bridge, according to all the evidence. For a distance of over 100 ft. it ran on the ties, bearing against the guard-rails here and there, with relatively little destruction to the track. But

at the point where the track leaves the bridge there begins a stretch over which the track was completely torn out. The inner guard-rails continued a hundred ft. or more beyond this point, but they could not save the track. The bunching effect of the impact of the wheels on the widely spaced ties was too great to be resisted.

The ties on the bridge, however, were bolted down to the supporting wooden stringers, and were thus anchored and spaced securely enough to hold. Great value is, therefore, to be assigned to secure bolting of ties to stringers or other means of securing ties against longitudinal displacement.

Another Record Broken

We believe it is customary in some departments at least of the government service to regard an appropriation of money as entailing a moral obligation to spend the full amount thereof. Since at the end of the fiscal year any balance reverts to the Treasury, it is necessary sometimes to put forth strenuous efforts to use up all the available funds; but the officials exert themselves with great energy to this end and usually succeed.

Quite in the same way it is the well-nigh invariable rule in private affairs that money which is collected by subscription for such "general welfare" work as the entertainment of conventions is spent in full. In fact, deficits are the usual outcome. Now, however, we are able to report a case of this class where a substantial balance was left over, and a "dividend" is to be returned to the subscribers. We believe this case establishes a new record. At least, if there have been earlier cases of dividends returned to the supporters of such undertakings, they must have been extremely rare.

During the past summer a considerable number of persons interested in testing materials subscribed money for the expenses of conducting the Sixth Congress of the International Association for Testing Materials, which was held last September. Subscribers were informed in advance that if any money was left over after paying the proper expenses of the congress it would be returned *pro rata*. The foreseen (but unexpected) contingency has been realized; the expenditures were kept below the receipts from subscriptions; and a distribution of the balance is soon to be made. The total collections amounted to something like $40,000, the expenditures (including a respectable subvention to the foreign society) were about $35,000, and there is a balance of $5000 for "dividends." This is a 12% rate of rebate.

We call attention to this financial record as a noteworthy achievement. Whoever has been connected with subscription undertakings knows that there is a multitude of claims and few "watch-dogs of the treasury," and knows also how urgent are the various demands for increasing expenses to the elastic limit so as to "eat up" or "lose" in an honorable manner any threatened surplus. That such claims and demands were withstood in this instance, and an actual money surplus is left for return, entitles the finance committee to high praise. May this record encourage liberal subscriptions in aid of future conventions of this association.

An Opportunity for Building the Greatest Bridge in the World

New York City has become the greatest metropolis in the world. London exceeds it, indeed, in the number of inhabitants included within the metropolitan area; but in the wealth of its inhabitants and the value of its commerce and manufacturing industries, New York has already outstripped its older rival. This could hardly be otherwise, for New York is the financial and commercial center of the most prosperous nation on the globe, whose wealth of natural resources is the envy of all the older nations.

The limits of Manhattan Island, on which New York City was founded, were long ago outgrown. The city has spread to the north across the Harlem River, and to the east across the strait connecting Long Island Sound and New York, known as the East River. Twenty-eight years ago the first Brooklyn Bridge, at the time the greatest bridge in all the world, was opened for traffic, and the swelling population poured across it to find new homes on the western end of Long Island. That expansion has continued at an amazing rate. Where that one East River bridge carried during the first year or so of its opening, 27 years ago, a comparatively thin traffic, there are now four bridges spanning the East River, and seven tunnels beneath it. Additional tunnels are planned and soon to be undertaken as part of the new underground rapid transit lines.

During all these years of phenomenal growth, however, the Hudson River, flowing down along the west shore of Manhattan Island and separating it from the entire continent to the west, from which it draws its chief commerce, has remained unbridged. Nowhere in the world does there exist an opportunity for a huge bridge structure which could command so great a volume of highly profitable traffic.

The story of the failure of previous attempts to bridge the Hudson at this point is a most interesting one. Readers of ENGINEERING NEWS unfamiliar with the local geography of New York should understand that the Hudson River for the lower twenty-five miles of its course till it expands into New York harbor, is bordered on the west by a continuous precipice of trap rock rising boldly from the water's edge—the famous Palisades. This high precipice, with the great width of the river and its great depth to rock bottom, preclude the possibilities of a bridge across the river at any point north of New York City until the Highlands are reached, a distance of some 40 miles. There is, in fact, no bridge across the river until Poughkeepsie is reached, 70 miles north of New York.

A quarter of a century ago, shortly after the success of the first Brooklyn Bridge had established the possibility of suspension bridges of long span, there was active interest in the project to bridge the Hudson River at New York. Two rival groups of financiers engaged in these projects. One group proposed to build a pier in the center of the North River so that the stream could be crossed by two spans, each about the length of the old Brooklyn Bridge. Permission to build this structure was refused by the War Department engineers, because of the obstruction to navigation which such a pier in the middle of the river would constitute.

Another project was originated by Gustav Lindenthal, who was then, as now, among the foremost bridge engineers of the country and who had to a high degree the confidence of prominent financiers. Mr. Lindenthal's plan was for a great suspension bridge crossing the river with a single span of some 3000 ft. His project was made public before the American Society of Civil Engineers and received the widest discussion. Approval of the War Department was necessary to the project, and a report upon the enterprise was made by Capt. Bixby (now General and Chief of Engineers, U. S. A.), which ranks among the classic documents of the engineering profession, especially in its study of wind pressures as affecting the feasibility of such a structure.

While, as always happens with any novel proposition in engineering, there was more or less difference of opinion in the profession, it can safely be said that the majority of engineers approved Mr. Lindenthal's design. The great cost of the structure, with its necessary terminals, however, made it essential that in order to be financially feasible the principay railways terminating on the west bank of the Hudson opposite New York City should unite in financing the great structure, or in agreement for its joint use.

Many of those who took part in the negotiations of that day are no longer living. The whole inner history of the negotiations may never be known. It can certainly be said, however, that had it not been for a little mutual suspicion, a little hanging back on the part of some who failed to see the necessities of the

near future, a little impatience on the part of others, the negotiations would have resulted in success and a great bridge across the North River would now have been in existence for many years.

Arrangements had been, in fact, concluded for financing the enterprise, conditioned upon the contracts for the rental of the structure; but the financial stress of the early '90's intervened and compelled this, with many other engineering enterprises, to be abandoned.

Following that period of financial stress, which characterized the last decade of the 19th century, there came a new period of expansion. The late A. J. Cassatt, President of the Pennsylvania R.R., determined that his system must have a passenger terminal upon Manhattan Island. The outlay necessary for bridging the North River, however, appeared too great for the Pennsylvania R.R. to undertake single-handed, and there seemed no better chance of cooperation between the different railways concerned than there had been ten years earlier.

About this time also there was a widespread idea that the solution of crossing wide and deep rivers was to be found in tunnels rather than in bridges. The success of the New York Rapid Transit subway demonstrated the feasibility of electric motive power for heavy passenger traffic, and was one thing which turned men's minds in this direction. Another was the successful completion of the tube tunnels for the Grand Trunk Ry. under the St. Clair River at Port Huron, Mich. The Pennsylvania's great rival, the New York Central System, was about undertaking its enormous task of rebuilding and electrifying its tunnel terminal into the Grand Central Station. Ideas also were afloat at this time concerning the extreme cheapness of submarine tunnels as compared with bridges of long span. There is probably little doubt that Mr. Cassatt, in making the momentous decision that the Pennsylvania should tunnel under the North River to a terminal on Manhattan Island, believed that he was undertaking for his company an enterprise which would give it the desired facilities at far less cost than would be involved in a bridge structure.

The accumulated experience of the last dozen years has demonstrated the fallacy of this assumption. The entire amount which the Pennsylvania has expended upon its new Manhattan Terminal, over $100,000,000, would in all probability have been sufficient to have created a bridge across the North River with a dozen or more tracks in place of its present tunnel crossing with two tracks. It is true that under such conditions as exist at the St. Clair tunnel, for example, a double-track tunnel can be run beneath a river at much less cost than would be required to construct a bridge; but a great bridge of large span could have a width and capacity sufficient to carry a dozen or twenty parallel tracks, while the Pennsylvania, for its $100,000,000 expenditure, has secured only two tracks under the North River and three tracks under the East River.

It is of public importance to raise the question at this time, for how many years longer New York City is to be permanently isolated by the North River on its western boundary from the continent lying beyond it. The city has indeed already expanded into New Jersey. A population of over half a million has its homes in New Jersey and earns its living in New York City. Tube tunnels under the river, built and operated by the Hudson Companies, now give access by a direct line to Newark and also to the railway stations in Jersey City and Hoboken; but the limited capacity of these tube lines to handle the rapidly increasing traffic is already evident. The Pennsylvania's two expensive tunnels under the river are used solely for through passenger traffic. They can accommodate at best only a trifling amount of suburban traffic, and it would hardly be attempted to operate freight trains through them.

The problem of the economic transfer of freight across the great river is one which grows more pressing every year. Projects for freight tunnels under the river have been agitated at intervals for a half dozen years past; but the great expense of such tunnels and the difficulty in connection with their terminals have prevented their financing.

In this number of ENGINEERING NEWS, Gustav Lindenthal, who originally designed and projected the great bridge across the North River twenty-five years ago, sets forth the reasons why the present is the opportune time for reviving this great project. It is important not only to the railway corporations concerned, but for the commercial welfare of New York City and the industrial welfare of its teeming population.

In the engineering profession one would look far today to find doubt that such a bridge is entirely feasible. The span required is no greater than that which is being boldly undertaken by the little city of Quebec across the St. Lawrence. The achievements and advances made in bridge engineering in the past quarter century, the monumental structure at the Frith of Forth, the great achievements in bridge engineering on the continent of Europe, with all that has been done in and about New York City itself, the advancement in the machinery for bridge construction and erection with the improvement in the quality of metal now available to the engineer—with all these great precedents now available, it will surely be agreed that to bridge the North River is well within the possibilities of American engineering.

The Royal Commission Standards for Sewage Treatment and for Sewage Disposal without Treatment

The new standards for sewage treatment and for sewage disposal without treatment announced a few weeks ago by the Royal Commission on Sewage Disposal are as revolutionary for England as they are rational, scientific and practicable for that or any other country. Casting to the winds the old notions long held by many and recently taken up in this country by increasing numbers of zealous but poorly informed people to the effect that all sewage should be "purified" before it is discharged into water and that a single absolute universally applicable standard is both feasible and imperative, the Commission has adopted the sound principle that the governing condition in sewage disposal is the prevention of nuisance. Nuisance depends upon the degree of dilution. Given sufficient dilution, no treatment of sewage is necessary. Nuisance conditions having been met, water-supply may, as a rule, be left to the water authorities and all other health conditions left out of account.

Too literal an application of what may conveniently be called the nuisance-dilution principle would result in no general standard but rather a separate standard for each works, and a changing one at that. This would be embarrassing from both the local and the central administrative viewpoints. Therefore, the Commission recommends a few standards, correlated with commonly used and successful methods of complete and of partial treatment, and beyond this a standard is provided for determining when no treatment whatever is needed.

Briefly, the standards are as follows: Where local conditions (chiefly a very low available dilution) demand complete treatment, the effluent shall contain not more than 3 parts of suspended solids per 100,000 and shall not take up more than 2 parts of dissolved oxygen per 100,000 after five days at a temperature of 65° F. (the test to be made without removing the suspended solids). Where partial treatment is all that is required, the standard is the amount of suspended solids in the effluent for varying degrees of dry-weather flow stream dilution (but dissolved oxygen calculations were used in arriving at these standards). For dilutions of 150 to 300 times, the suspended solids in the works' effluent shall not be more than 6 parts per 100,000, which can readily be attained by chemical precipitation; for dilutions of 300 to 500 times, the suspended solids shall not be more than 15 per 100,000, which can be attained by either plain sedimentation or septic tanks. For di-

lutions greater than 500 times, no treatment would be necessary, except such screens or detritus tanks as the "Central Authority" might require.

It should be understood that the highest of the standards cited is to be deemed a "normal standard," of general application except as a "relaxed standard" is authorized by the Central Authority; also that the normal standard, according to a formula used by the Commission in its calculations, is based on a dilution as low as eight volumes of dry-weather stream flow to one volume of effluent, but that exemptions from the normal standard would not be permissible until 150 dilutions were reached.

In considering these standards, American engineers should remember that the average stream in England is small compared with our own streams, and that as a rule British sewage is much stronger than American sewage. Allowance should be made for these and other varying conditions in considering how the British standards may be utilized here.

The basic principles underlying the standards recommended by the Royal Commission on Sewage Disposal are in complete accord with the ideas and practice of most of our best American engineers. The standards themselves, however, seem to be far in advance of any yet proposed for general adoption in any country—not in severity but in point of scientific analysis of the sewage-nuisance problem and in correlation of nuisance prevention with degrees of dilution and degrees of treatment.*

*Although we expect to deal with this valuable report at greater length in a later issue those of our readers who are following sewage-disposal problems closely will, of course, wish to see the full report, and possibly the appendix, not yet available.

The report proper comprises less than twenty pages and is published in London at only 3d. or about 6c., by Wyman & Sons, Fetter Lane, E. C., London, England. Probably 1d. extra would be charged for postage to the United States. We are not yet able to state the price of the appendix, but it is likely to be several shillings. Those who wish to order direct can best make remittances by international money order.

Annual Convention of the National Association of Cement Users

The ninth annual convention of the National Association of Cement Users was held at Pittsburgh, Penn., Dec. 10-14, 1912. There has been considerable complaint in the past that the annual conventions contained too many sessions and extended over too long a time, so this year the usual nine sessions from Tuesday morning through Saturday night were changed to eight sessions beginning Tuesday night and lasting through Friday night. As the opening session included only one technical paper, most of the members present did not arrive until Wednesday morning. The attendance was measurably lower than in any of the conventions of late years, the registration being under 150.

The technical proceedings were, as usual, divided into four parts, topical discussion of subjects of interest to concrete men, committee reports, illustrated lectures and technical papers. The topical discussions, although scheduled, were conspicuous by their absence, but some seven committee reports were presented, eleven lectures delivered and twelve technical papers read. These are all briefly referred to below.

The convention program also allowed time for a visit to the local cement show, attendance on a most successful banquet and on Saturday morning a visit to the Pittsburgh laboratory of the U. S. Bureau of Standards, where a 30-in. concrete column was tested to destruction and where the numerous other features of the plant were thoroughly inspected.

Committee Reports

The following committees brought in reports either of progress or containing definite recommendations:

Reinforced-concrete Highway Bridges and Culverts (Willis Whited, Chairman)—The chairman of this committee, which was only appointed last year, read a progress report prepared by himself, submitting certain tentative clauses of a proposed specification governing the design of concrete highway bridges. It was received as a progress report to be commented on by the membership for the benefit of the committee in preparing final specifications.

Concrete Surfaces (L. C. Wason, Chairman)—This committee submitted a final report on "Portland Cement Stucco," embodying a number of changes in the tentative report presented last year. There was a lively discussion relative to the inclusion of a specification for a wood-lath backing for the stucco and to the proposed specification of a lime content in the cement stucco. In view of the fact that no member of the committee or of the association on the floor felt competent to pass on the question of the wearing qualities of stucco on wood lath, it was decided to omit that part of the specification this year. Some floor members contended that the lime content in stucco was merely a concession to old-time methods and to the wishes of the masons, who want an easy working material, and advised that the specification should specify as preferable a stucco containing no lime but merely cement. These features are to be embodied in the specification which was accepted and is to be sent to letter ballot.

Reinforced Concrete and Building Laws (A. E. Lindau, Chairman)—This committee has done no definite work during the year, but reported that it intends to make load tests the coming year on 20-in. reinforced-concrete columns and fire tests on reinforced-concrete mullions.

Measuring Concrete (R. C. Cummings, Chairman)—This committee presented a final report, commenting upon some of the points brought up by its first report of last year and embodying an estimate of a typical building according to the principles laid down there. The report was accepted for letter ballot and the committee discharged.

Roadways, Sidewalks and Floors (C. W. Boynton, Chairman)—This committee submitted certain minor changes in the specifications on floors, curbs, gutters, roads and pavements, all of which were accepted and passed to letter ballot.

Methods of Tests (S. E. Thompson, Chairman)—The Committee on Standard Specifications and Methods of Tests of Concrete Materials presented a very valuable progress report embodying a suggested scheme for the uniform testing of concrete materials. The major portion of the paper will appear in an early issue of Engineering News.

Aggregates (W. M. Kinney, Chairman)—This committee has just been organized and could report only that it intends to lay out a definite program looking toward the acquisition of some definite data on the aggregates used in concrete.

Illustrated Lectures

A very large part of the time was devoted to illustrated lectures upon subjects of interest to concrete users. While these lectures were very much appreciated by the members present, they were all upon subjects which demand more than the brief abstract possible to be given here, so we note below only the names of the speakers, with a short reference to their talks.

"The Use of Concrete in Irrigation Work," by Arthur P. Davis, Chief Engineer of the U. S. Reclamation Service—An illustrated description of a great number of concrete dams, canals, conduits, sluiceways and other structures built by the federal government in the development of the arid West.

"The Concrete Roads of Wayne County, Michigan," by Edward N. Hines, Road Commissioner of Wayne County—A series of moving pictures illustrating the construction of an extensive mileage of one-course concrete roadways constructed during the past two years in and around Detroit, Mich. These roads are built by day labor under the direction of the county, which has provided a large construction and storage plant and is prepared to continue the work each year in increasing quantities. The results have been most satisfactory. The roads are laid on a rolled and packed flat earth

footing, about 7 in. thick on the sides and crowned ¼ in. to the foot. A 1:1½:3 concrete is used. The costs vary from $1.05 to $1.75 per sq.yd.

"The Use of Concrete by the Bureau of Yards and Docks, U. S. Navy," by Rear Admiral H. R. Staniford, Corps of Civil Engineers, U. S. Navy—A paper, read by Civil Engineer Carlson, U. S. N., describing mainly the several large concrete dry docks built by the government. It has been found necessary to line these docks with stone or brick, since the varying tides and temperatures seriously affect the integrity of the concrete work. Some buildings of reinforced concrete were also shown.

"Reinforced Concrete in Railroad Work," by M. A. Long, Architect, B. & O. R.R.—A series of illustrations, with comments, of various buildings built of concrete by some of the Eastern railroads. The paper took up the subject mainly from the viewpoint of the architect and suggested some of the features of concrete construction, in which the architect could desire improvement, mainly surface treatment, floor dusting and waterproofing.

"Application of Concrete in the Abolition of Grade Crossings," by James W. Phillips, Assistant Engineer, Department of Public Works, Philadelphia, Penn.—An illustrated account of a number of grade crossing concrete structures built by the railways entering Philadelphia.

"Some Recent Applications of Concrete in Railroad Work," by Frederick Auryansen, Bridge Engineer, Long Island R.R. Co.—A description of some walls, bridges and station crossings recently completed by the Long Island R.R. in the new work entering Brooklyn and Manhattan, New York City.

"Some Recent Applications of Concrete and Reinforced Concrete in the City of Pittsburgh," by Norman S. Sprague, Superintendent, Bureau of Construction, Department of Public Works, Pittsburgh, Penn.—An analysis of the problems of municipal bridge construction, illustrated by reference to the numerous large concrete bridges recently built in Pittsburgh, some of which are presented in the first article in this issue.

"The Manufacture of Concrete Products in Germany," by Julius Carstanjen, Duisberg, Germany—A lecture on the development of the art of making artificial stone in Germany by a member of one of the largest German firms engaged in that business. This industry has developed to a remarkable degree and now includes the molding of the most complicated statuary as well as the usual decorative and architectural pieces. Two notable contracts described were the substitution of concrete for stone that had weathered to a point of failure in the bases of the exterior columns in a government building and the substitution of concrete for stone heroic statues on another building. In this latter case the stone statues were used to make the molds in which the concrete replicas were cast.

"The Coloring and Texture of Concrete Products," by Adolph Schilling, Philadelphia, Penn.—A description of American methods in casting artificial stone.

"The Present Status of Unit Methods of Reinforced-Concrete Construction," by John E. Conzelman—A paper read by Charles D. Watson, describing the present development of the unit system of precasting and erecting into place separate concrete members of a structure. Over 40 acres of such construction are now under contract.

ANNUAL ADDRESS OF THE PRESIDENT—The annual address of President Humphrey consisted in a talk, illustrated by slides, on the use of concrete in European mining practice.

PAPERS

In addition to the above-noted reports and lectures the following papers were read:

REINFORCED-CONCRETE PATENTS—W. M. Denman, Springfield, Mass., read a paper entitled "Patents Applying to Reinforced Concrete Design and Construction," describing what he chose to consider the most important patents granted on concrete structures by the U. S. government. Excluding all devices relating to the making and mixing of concrete, it seems that the number of United States patents on concrete structures has increased from about three per annum, 10 years ago, to over 150 per annum today. Mr. Denman selected as the only important ones of this number about 20 patents held by a half-dozen men and described them briefly and indicated the number of infringement suits in which each has been upheld. His attitude was frankly in favor of upholding these patent rights and he urged all engineers to utilize the patented ideas, paying the proper royalties, of course, in all work where they could be applied. In answer to a question be stated that he considered it impossible to design or construct an economical and at the same time a safe reinforced-concrete bridge without utilizing or infringing upon certain patents. This radical answer seemed to confirm the general impression of decided bias on the speaker's part and there was no further discussion.

ELECTROLYSIS OF CONCRETE—A very important paper by three members of the U. S. Bureau of Standards was read by one of the authors, Burton McCollum. It is entitled "The Effect of Electric Current on Concrete," and is by E. P. Rosa, Burton McCollum and O. S. Peters. A full abstract appears on another page of this issue. Mr. McCollum stated that the studies on which the paper is based will form the subject of a much more complete bulletin to be issued soon by the bureau.

FIRE TEST OF PLASTERED PARTITIONS—The tests on test walls made up of stucco on plaster board, wire lath and wood lath made at Cleveland, Ohio, and described in ENGINEERING NEWS, Aug. 29, 1912, p. 403, were the subject of a very full paper by L. H. Miller, Cleveland, Ohio, and a discussion by H. B. McMaster. Those interested are referred to the issue of ENGINEERING NEWS noted.

TESTS OF WATERPROOFING FOR CONCRETE—In a paper with the above title, Cloyd M. Chapman, New York City, outlined the undulying principles which should govern such tests. The keynote of his advice was that the tests should approximate as near as possible the conditions of the structure on which the waterproofing is to be used, that is the same material, method of manufacture and condition of service should be attempted. He particularly referred to a special test-piece which he had devised in which a projecting piece is broken off, leaving a rough, natural surface on which to make the test.

TESTS ON FLOORS—Two papers on load tests of floors in completed buildings were presented, one by T. L. Condron, Chicago, Ill., entitled "Principles of Design and Results of Tests on Girderless Floor Construction of Reinforced Concrete," and the other by Arthur R. Lord, Chicago, Ill., entitled "Measurement of Actual Stresses in a Cantilever Flat Slab Reinforced-Concrete Floor Having Rectangular Panels." Mr. Condron's paper described some load deflection tests on buildings in Chicago to satisfy the Building Department there as to the strength and design of the system of which he is the inventor. He protested strongly against the law which requires a test load of twice the live load plus the dead load, on the ground that so great an excess over design load might work serious injury to the structure. Mr. Lord's tests were similar to those he has described in previous conventions of the association, in that the elastic deformation of concrete and steel was measured under load. We hope to present later an abstract of the tests.

RIGID FRAME ANALYSIS—Sanford E. Thompson, Newton Highlands, Mass., presented an analysis of the stresses in concrete structures, considering whole sections of the structure as a rigid frame. This demands a study of the elastic deformation of the various parts acting in relation to each other and involves some complicated mathematical treatment. The speaker said that the final forms to which the equations could be reduced were not so complicated as to prohibit their use in practice and that such use would often allow of a more economical disposition of the material than if the regular formulas were followed.

LATERAL DISTRIBUTION OF STRESSES—Prof. W. A. Slater, University of Illinois, presented a long paper entitled "Tests to Determine Lateral Distribution of Stresses in Wide Reinforced-Concrete Beams." These tests were made on beams varying in width up to 72 in. and an analysis was made into the manner in which the loads were carried. An abstract of the paper will appear later.

BUREAU OF STANDARDS' WORK—Two members of the U. S. Bureau of Standards described the work of that bureau in concrete and cement fields. P. H. Bates, Pittsburgh, told of the studies now being made into the constitution of portland cement and presented a number of slides showing microphotographs of cement clinker. An effort is being made at the laboratory to supplement the studies made at the Geophysical Laboratory at Washington and to establish, if possible, some satisfactory theory of the composition and action of portland cement. Rudolph J. Wig (Washington, D. C.) outlined the proposed work of the bureau and submitted a record of what it had already accomplished in cement and concrete work.

BUSINESS AFFAIRS

The present year seems to be a critical one in the history of the association. The Board of Direction reported that the expenses are running far in excess of the income, so that there is now a deficit so large that it has not felt warranted in authorizing the publication of the *Proceedings* of the convention held in Kansas City last March. This deficit has been brought about by the increase in the services of the association greater than warranted by the increase in membership. An office has been opened at Philadelphia with a salaried, though small, force, and the effort has been made to issue the standard specifications and letter ballots of the association in short order. At the same time there have not been as many new members as might reasonably be expected and the Board of Direction has authorized the dropping of a number of old members for non-payment of dues. In consequence there are actually fewer members today than a few years ago, with consequently reduced receipts.

The board in its report frankly recognized this condition and reported that in order to better financial conditions, it was proposed to solicit contributing membership in sufficient number to provide an annual income of $6000 which, in addition to the income from the ordinary $5 membership, would serve to put the society on its feet. It also reported that it felt confident that such subscriptions would be received from the industrial interests concerned with the development of good concrete.

A further radical change which was much discussed was the proposal to issue a monthly journal of the association, which journal should contain in the 12 issues the same material now appearing in the *Proceedings*. It was thought that the interest produced by the regular and prompt issuance of the very valuable material presented to the convention and other material available to the association, would soon increase the regular $5 membership to such proportions that there would be no need to solicit the larger contributing memberships. This proposition appears to be very favorably considered by the board, and it is very probable that January or February will see the first number of such a journal. The Kansas City *Proceedings* will be published in one volume, as heretofore.

OFFICERS—For the ensuing year Richard L. Humphrey was again elected President, with Prof. A. N. Talbot and L. C. Wason, as Vice-Presidents. E. E. Krauss is Secretary, with offices in the Harrison Bldg., Philadelphia, Penn.

Electrolysis of Concrete*

By E. B. Rosa†
Burton McCollum†
and O. S. Peters†

Results are given of the most comprehensive series of laboratory and field studies of the effects of electric currents in plain and reinforced concrete yet reported. Previous investigators have failed to find damage with current flowing from concrete to iron; but it occurred in these tests. Disruption of specimens with heavy current from iron to concrete was obtained as before but without softening of the concrete. Slight currents were endured for a year and a half without damage to normal specimens but the presence of chlorides always facilitated trouble. Preventive measures for actual structures are discussed.

*Extracts from a paper before the National Association of Cement Users, Pittsburgh, Dec. 10-14, 1912.

†U. S. Bureau of Standards, Washington, D. C.

Following the early demonstrations of the possibility of damage to concrete by electric currents, reports of serious damage to certain concrete buildings, bridges, etc., become current, and considerable apprehension has been aroused in some quarters that great damage may be in progress due to this cause. The subject was brought directly before the Bureau of Standards by numerous letters of inquiry from engineers, contractors, and corporations requesting information in regard to the probable extent of the damage and the most feasible methods of preventing it. Recognizing the great practical importance of the subject, the Bureau of Standards has undertaken a thorough investigation into the cause and nature of the phenomena.

The work was begun during the summer of 1910 and certain phases of the investigation are still in progress, but enough work has been completed to justify the publication of a report of progress at this time. The investigation has comprised three general branches, as follows: (1) Laboratory investigations relating to the cause and nature of the phenomena caused by the passage of electric currents through concrete; (2) investigations in the field with the view of establishing definitely the probable extent of the danger in practice and the circumstances under which trouble is most likely to occur; (3) a study of the various possible means of mitigating trouble from this source, leading to specific recommendations based thereon.

Laboratory Investigations

FORM AND COMPOSITION OF TEST SPECIMENS—At the outset, a number of the experiments described by previous experimenters were repeated in order to verify their results and to afford an opportunity for studying at first hand the phenomena previously observed. At the same time numerous modifications of these experiments as well as many radically different ones, designed to throw special light on particular phases of the subject, were instituted. In this preliminary work one general type of specimen was adhered to as far as practicable. A number of well known brands of portland cement, all of which conform to the requirements of the standard specifications for portland cement, were selected, together with sand and stone similar to those used in large quantities by builders in the city of Washington for reinforced-concrete construction work. Using filtered Potomac River water, these ingredients were made into a 1:2½:4 concrete; a proportioning which gave a very dense aggregate. In many special experiments, especially where chemical analyses were to be made, quartz sand and distilled water were used.

The specimens were moulded as cylinders 6 in. in diameter and 8 in. long, with a 12-in. electrode embedded 7 in. on the axis of each cylinder. The specimens were allowed to remain in the molds until

set sufficiently for handling and they were then removed and buried in wet sand for approximately twenty days. At the end of this time they were taken out of the sand and placed in the shelves of a case, where they were wet down occasionally until used.

Where iron served as the embedded electrode it was cleaned of scale and rust by pickling in dilute sulphuric acid, dipped in limewater, numbered, weighed and immediately embedded. This insured entire absence of rust to begin with. Electrodes other than iron were sandpapered or otherwise cleaned to a bridge surface before embedding.

ARRANGEMENT OF SPECIMENS IN VESSELS—Each specimen put under test was placed in an earthenware jar 8½ in. in diameter and 9 in. deep. For an outer electrode an 8x24-in. piece of No. 24 gage sheet iron was rolled into a cylinder 7 in. in diameter by 8 in. long and placed around the specimen. The jar, with the exception of cases otherwise designated, was filled to about 1 in. from the top of the specimen with tap water. Rubber-covered wire leads were soldered to the outer sheet-iron electrode and to the projecting end of the embedded metal. By connecting these leads to the proper poles of the electric circuits the embedded metal could be made either anode or cathode.

DIVISION OF WORK—The description of the phenomena observed is given under two heads, (1) anode effects, or those effects observed where the current flows from the embedded iron out into the concrete, and (2) cathode effects, or those occurring when the current flows from the concrete to the embedded iron. In both cases experiments were carried out under both high and low voltages, and as the results were radically different in the two cases, they are discussed below under separate heads. In the high-voltage tests the specimens were subjected to voltages varying from about 50 to 70 volts, and the current flow was continuous except for infrequent and irregular intervals when work was being done on the specimens under test or on the electric circuits to which they were connected. The time during which they were disconnected amounted to less than 1% of the total time. In the low-voltage tests the current was maintained for 7½ hr. each working day for the first eight weeks, after which the current was on for 24 hr. each day with the exception of Sundays and holidays, and an occasional short interval when the machine was shut down for repairs. In calculating the number of hours of a test only the time during which current flowed was considered.

ANODE TESTS ON HIGH VOLTAGE—Only a comparatively small number of specimens were tested on high voltage because such conditions are abnormal and would rarely, if ever, be encountered in practice. A few of these were made, however, to check the observations of previous experimenters and the results noted in these tests are recorded briefly below. In the preliminary tests on high voltage, in which iron served as the anode, eleven specimens were used. They were all of like proportions, viz., 1:2½:4, and were connected two in series on 115 volts d.c.

All of the specimens behaved in substantially the same manner. During the first few hours there was an appreciable rise in the external temperature. In most cases this rise amounted to from 12° to 25° C. above room temperature. After a few hours of current flow bubbles of water containing iron rust began to appear around the anode and cracks soon developed in the concrete. The manner in which the cracking occurred was the same in all specimens. First, there appeared a very fine crack beginning at the iron and extending outwardly in practically a radial direction. This crack soon extended to the outer surface of the concrete cylinder, and then gradually widened, while at the same time two or three other and smaller cracks appeared

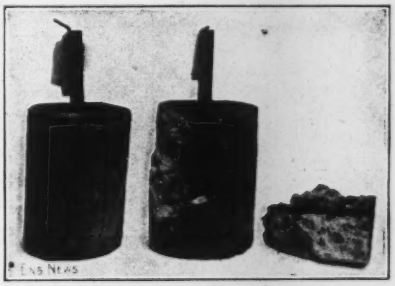


FIG. 1. TYPICAL CRACKED IRON-ANODE SPECIMENS IN U. S. BUREAU OF STANDARDS STUDIES OF ELECTROLYTIC DESTRUCTION OF CONCRETE; HIGH-CURRENT DENSITY TESTS

on the opposite side of the anode from the initial crack, and also radiating from the center. Most of the specimens cracked in about the same time, the first eight in 23 hr.; two containing another brand of cement in 72 hr., a third made of a third brand in 96 hr., but under special conditions.

The manner in which these cracks occurred and developed was strongly suggestive of a wedgelike action slowly applied at the center of the specimen. The specimens were readily pried open and separated into several pieces. The individual pieces of concrete thus obtained were, however, to all appearances as strong as similar specimens of concrete that have not been subjected to the influence of electric currents, and were broken with a hammer with the same difficulty. There is no indication whatever that the cement decayed or deteriorated in any way as reported by some earlier investigators, but on the contrary the appearance of the concrete, and its mechanical properties, both indicate very strongly that there has been no such action.

The variation of the electrical resistance of the specimens with time is of particular interest. The first eight specimens, all containing one brand of cement, show a fair degree of uniformity in initial resistance, the average being 68 ohms. At the end of 23 hr., when cracks first appeared, the average was 133 ohms. At the end of the test this average had risen to 2194 ohms, or over 32 times its initial value. This enormous rise in resistance with time is of the greatest importance, particularly from the practical standpoint. The current varied in these specimens from 0.57 to 0.79 amp.

On breaking open the specimens the embedded iron was found to have been very badly corroded, thick layers of scale consisting largely of the oxides of iron having formed all over the surface and to some extent in the voids in the concrete adjacent to the iron. In those specimens which were left in circuit for a long period after cracks developed, the deposit of oxides spread for some distance out into the cracks, but in specimens which were removed and opened immediately after cracking, this was not the case. Upon breaking open the solid portions of the concrete, it was found that the discoloration due to the products of corrosion had not penetrated into the body of the concrete more than ⅛ in. or so.

The general appearance of the specimens is shown in Fig. 1.

ANODE TESTS ON LOW VOLTAGE—Tests on specimens subjected to 15 volts and less were carried out in much greater number than those on high voltage. At the outset of the investigation 90 specimens containing iron electrodes were placed in circuit and watched for a period of about 5500 hr. (7½ mo.), during which time careful records were kept of voltage and current flow. Now and then a few specimens were broken open to give an idea of what was taking place. Finally, a large number were broken open, the amount of the corrosion determined, and the general condition of the concrete noted. A most conspicuous feature of the results of this test, and a very surprising one, in view of the results previously obtained at higher voltages, is the fact that cracking almost universally failed to occur. Of the ninety specimens under test only three had cracked at the end of 5500 hr.

It seems allowable, therefore, to say that not one of 87 normal specimens made of ordinary portland cement and put under test on 15 volts or less (currents of 0.0022 to 0.0191 amp.) had cracked at the end of 5500 hr. Of the specimens which were then opened for examination it was found that in every case the concrete was broken with difficulty and appeared to be as sound as that

of similar specimens made at the same time and not subjected to the action of electric currents. In practically all cases there was more or less corrosion around the anode at the point where it entered the concrete and in some instances for an inch or so below the surface. In some cases, also, where there were voids in the concrete next to the iron there was some rust and slight pitting, but in nearly all cases where the concrete was in actual contact with the iron the corrosion was quite small and in many cases the iron was practically as bright and clean as when placed in the concrete. It is important to note that the total number of ampere-hours per square inch of embedded electrode surface carried, on the average, is considerably larger than the corresponding figures for the high-voltage specimens at the time they cracked, the average being 2.6 for the low-voltage specimens, as against 0.83 required to crack the high-voltage specimens. It is evident, therefore, that the quantity of electricity that passes through a specimen does not alone determine the amount of damage that it may do, but that the rate at which the current flows is also an important factor. Moreover, it must be evident from these observations that the rate at which damage occurs decreases with decreasing voltage much more rapidly than the voltage is lowered, since in the present instance a reduction of the voltage to one-fourth of the value used in the high-voltage tests enabled the specimens to run with little or no damage for a period over two hundred times as long as was required to destroy completely the specimens on the higher voltage.

TESTS ON OTHER-THAN-IRON ANODES—A number of specimens were made up in a 1:2½:4 mixture, but using copper, brass, carbon and copper-clad steel as electrodes. Part of these were placed in circuit with the metal or carbon as anode, and subjected to voltages ranging from 15 to 60, for periods varying from a few days to ten months. There was in all cases considerable corrosion of the imbedded metal, but in no instance was the concrete cracked or otherwise injured. The fact that the concrete did not crack in any of the above cases is significant, and it is interesting to note in this connection that in the case of the metals used there is a tendency for the formation of soluble salts as the end products of corrosion. These remain in solution to some extent, diffuse and do not give rise to local mechanical pressure. This phenomena supports the theory that the cracking of the concrete is due to mechanical pressure developed by the formation of insoluble oxides at the anode surface.

Since the possibility had been suggested of using a protective film formed over the surface of an aluminum-covered rod, when embedded in concrete, tests were made on aluminum rods with voltages ranging from 5 to 115, the embedded metal being made anode in some cases and cathode in others. In all cases there was rapid deterioration of the aluminum. In the cases in which the aluminum was positive there was corrosion of the metal, accompanied by a swelling action which in three of the five specimens was sufficient to crack the blocks. When the aluminum electrode was made negative on 15 volts the specimen cracked at the end of 144 hr. and when opened, the metal was found to be badly pitted, but the corrosion was different from that found in the positive specimens. The decomposition products were black, forming a hard shell around the electrode from which they were separated very readily. This corrosion was doubtless due to a secondary action, and in the light of experiments described later it seems quite probable that the action is the result of a concentration of alkali metals (sodium and potassium) near the negative electrode with the resulting formation of sodium and potassium aluminates. The dark color is doubtless due to the presence of iron as impurity. A check specimen made at the same time as the others and kept in water without current flow showed no corrosion whatever with the exception of a slight reaction between the metal and the alkali in the cement forming a very thin, hard layer on the surface.

CATHODE EFFECTS—In the preceding section the phenomena noted are those resulting when current flows from the embedded iron or other metal out into the concrete. When the direction of current flow is reversed, making the iron cathode, very different effects are produced. In this case there is no tendency for the iron to corrode because of the current flow, but on the contrary the iron is protected from any natural corrosion that might tend to take place. The conclusion has been widely accepted that when the current flows from concrete to iron no effects are produced; at the time these experiments were begun there appeared to be no substantial ground on which to question this conclusion. It was deemed advisable, however, to confirm these observations, and accordingly a number of specimens were made exactly similar to those used in the anode tests above described, using not only iron, but also brass, copper and carbon as electrodes. These were placed in circuit with the current flowing from the concrete to the electrode. Both high and low voltages were used, and the conditions in general were kept exactly the same as with the anode tests except for the direction of current flow. There was a rise in resistance as the test proceeded, very similar to that which occurred in the anode specimens, but this rise was less marked, the average ratio of increase being about 10 to 1. Throughout the tests with iron cathodes, water was forced out around the embedded iron and kept the top of the specimen wet. This forcing out of water was evidently due to the formation of gas at the cathode, which, on escaping, forced the water through the pores of the concrete to the surface. This water carried calcium hydroxide in solution and as evaporation took place, calcium carbonate was deposited in rings surrounding the cathode on the surface of the concrete. The greater part of the gas evolved was hydrogen. As the test proceeded the mortar on the top surface of the specimen immediately surrounding the cathode became quite soft. This softening extended in the surface to a distance of about ¾ in. from the cathode and in this region the concrete was darker than elsewhere.

After the expiration of several months, and at intervals thereafter, certain of the specimens were broken open for examination. In every case the concrete blocks were broken with difficulty, the main body of the concrete being apparently as sound as in similar specimens not subjected to the action of electric currents. On laying the specimen open it was found that the embedded metal was in a perfect state of preservation, but the entire region surrounding the cathode for a distance of ⅛ in. to ¼ in. from the surface of the metal was considerably darker in appearance than the main body of the concrete and was very soft like the concrete immediately surrounding the cathode at the surface. The cement here could be shaved off with a knife like soft soapstone.

With copper and brass cathodes, respectively, the same general phenomena were observed. Both cathodes were quite slippery when first removed from the concrete. With iron cathodes subjected to only 15 volts throughout the test, the results were the same as above, except not advanced quite as far. One specimen contained as electrode a piece of arc-light carbon. The carbon itself disintegrated badly and after its bond had been destroyed it was forced out of the concrete by the formation of gas underneath. The mortar near the carbon was found to be disintegrated in the same manner, and to about the same extent as in the case of metal cathodes on 15 volts.

In all of the above specimens there was a rather sharp line of demarcation between the softened area and remainder of the concrete, the soft portion being in every case readily distinguishable by its darker color. Outside this darkened zone the concrete appeared to be as sound as in specimens not carrying current and no physical change of any kind could be detected therein. After the specimens had been broken open for some time and allowed to dry, the darkened zone became somewhat lighter in shade, but there always remained a distinct difference readily detectable by the eye. The soft-

ening also diminished greatly as the specimen became drier, and after becoming thoroughly dry it became nearly as hard as the unaffected mortar, but remained distinctly more friable.

BOND TESTS OF CATHODE SPECIMENS—The disintegration of the mortar in the immediate vicinity of the cathode, as described above, led to tests on the relative shearing strengths of the bond where current had passed with embedded iron cathode and where it had not. The bottoms of the saturated specimens were ground perpendicular to the axis of the embedded iron rods until the lower ends of the rods were exposed. The projecting ends were sawed down to 2 in. and the specimens placed, one at a time, in an Olsen testing machine which pushed the rods through the blocks sufficiently (about 0.1 in.) to give the maximum shearing strength of the bond and also the friction load after the bond was broken.

Under severe electrical conditions the bond was reduced to about one-fifth of its original value. Other specimens which were run at voltages ranging from 5 to 15 as described, showed the same disintegrating action; the extent of the softened area, while somewhat irregular and slightly indefinite, indicated that at least during the first few months of a test the amount of the disintegration is probably roughly proportional to the quantity of electricity that has passed through the concrete, and is thus but indirectly affected by the voltage used. In this respect it is in marked contrast to the anode effects previously described, which appear to diminish much more rapidly than the voltage, until at voltages of 5 to 10 on the specimens used they practically disappear.

CAUSE OF DISINTEGRATION ABOUT CATHODE—A number of experiments were carried out to determine the cause of the softening of the concrete near the negative terminal. It was found that all of the specimens contained considerable amounts of sodium and potassium in soluble form. When current passed through the specimen these alkalies were gradually concentrated near the negative terminal and in time the concentration of the alkali became sufficient to attack the cement, the principal reactions being the replacement of the calcium in the calcium silicates and aluminates by sodium or potassium with the formation of soluble sodium or potassium silicates and aluminates, and the formation of calcium hydroxide. Figs. 2 and 3 show the general appearance of the specimens after being broken open. As indicated by the cards accompanying specimens, some were made up with NaOH and KOH added to the water used in making the mortar. The percentages are expressed in terms of the amount of cement used.

EFFECT OF CURRENT ON THE MECHANICAL STRENGTH OF NONREINFORCED CONCRETE—While the foregoing experiments appear to show quite clearly that no effect is produced by the passage of an electric current through the main body of the concrete remote from the electrodes, it was deemed advisable to corroborate this with a series of mechanical tests on specimens so designed as to eliminate the electrode effects.

Concrete cylinders 6 in. in diameter by 8 in. long, with no iron embedded, were made of 1:2½:4 concrete, using one brand of cement for part and another for the rest. Sand and crushed trap rock constituted the remaining aggregate. After the concrete had become thoroughly set, 18 of the cylinders were provided with electrical connections by plastering 3x3-in. carbon plates to the ends of each cylinder with cement mortar; a copper lead was attached to each carbon plate. When the mortar, which held the electrodes on had set, the cylinders were laid on 2x2-in. strips of wood and the copper leads connected to the terminals of a 115-volt direct-current circuit. The potential gradient impressed on these specimens was therefore 172 volts per foot and the test was extremely severe. Check specimens were placed in the immediate vicinity and all of them were thoroughly wet down each day with tap water so both the check specimens and those carrying current were under the same moisture conditions. Current flowed continuously for 14 months and averaged 00.009 amp. for each specimen. This means that approximately 75 amp.-hr. passed through each specimen under a potential gradient of a little more than 14 volts per inch. The current density averaged about 0.0003 amp. per sq.in. cross-sectional area of specimen.

FIG. 2. NORMAL CONCRETE FIG. 3. CONCRETE CONTAINING SODIUM HYDROXIDE

FIGS. 2 AND 3. TEST SPECIMENS, SHOWING CATHODIC DISINTEGRATION OF REINFORCED CONCRETE; U. S. BUREAU OF STANDARDS

At the end of 14 months the specimens were removed from the circuit and tested for their crushing strength in a 200,000-lb. Olsen testing machine. Nearly all of the failures were of such a character as to show the usual 45° shear. There is no indication there that the passage of current seriously affected the crushing strength of the concrete.

CAUSE OF CRACKING OF REINFORCED CONCRETE BY CURRENT—We are convinced, however, that the theory which attributes the cracking to the formation of iron oxides between the iron and the concrete is the correct one. Of the numerous experiments which we have carried out to show this we shall describe but one here.

A cast-iron shell 1½ in. in external diameter was slotted lengthwise at opposite extremities of a diameter to a depth $\frac{1}{16}$ in. less than the thickness of the shell. The thickness of the shell was ¼ in. in this case, so the cuts were each $\frac{3}{16}$ in. deep, leaving a thickness of

⅛ in. of cast iron holding the two halves of the shell together on either side. The object in making slots of this character in the shell was to cause practically all of the elongation of the iron due to an internal pressure to take place opposite the slot. The smaller cross-sectional area of iron there would then reach its elastic limit sooner than the thicker portion and lead to a quick break in the event of the pressure becoming great, while a shell ⅛ in. in thickness throughout might expand to such an extent that a break would not occur. A ¾-in. solid-iron electrode was cemented in the bore of the shell with 1:1 mortar made up with a 3% NaCl solution. After the mortar had set, the shell was put in circuit on 15 volts with the central electrode positive. In about three days the two halves of the shell were found to be broken apart by the formation of rust on the surface of the positive electrode.

The area of the broken section of cast iron was found to be 0.106 sq.in. and its modulus of rupture about 20,000 lb. per sq.in. The actual pressure developed on the surface between the mortar and the rod at the time of rupture was calculated to be 3760 lb. per sq.in. This pressure is much more than ample to produce the cracking of concrete that has been observed in past experiments.

RISE OF RESISTANCE IN ANODE SPECIMENS—The electrical resistance of concrete is a factor of great importance but one which varies so greatly with varying physical conditions, especially with varying moisture content. An average figure for representative specimens of thoroughly wet concrete may be taken as varying from 4000 to 6000 ohms per centimeter cube. With reduced moisture content the resistance rises rapidly, the concrete becoming a fairly good insulator when thoroughly dry.

Previous investigators have mentioned that when electric current passed through reinforced concrete in the laboratory, there is a gradual increase of resistance with time, regardless of the direction of flow of current. Nothing definite has been said as to the probable cause of this rise of resistance, nor does it appear that its importance as a factor in minimizing trouble from electrolysis in reinforced concrete has been fully appreciated. With a few exceptions, which are considered later, such a rise of resistance occurred in all of the specimens tested in the course of this work. When the specimens were first placed in circuit the resistance was a minimum, but upon the application of the electric current the resistance began to rise. In the low-voltage specimens, the first apparent increase in resistance may have been more or less affected by polarization, but in the higher voltage tests at 15 to 100 volts, polarization effects are negligible. The resistance at first increased quite rapidly, but the rate of increase usually commenced to diminish after about 1000 to 1200 hr. The rise of resistance continued for a period varying from several days to several months, according to circumstances, until an approximately constant condition was reached. In the case of higher voltages the increase of resistance was the more rapid at first, but the rate of increase was not maintained in proportion to the voltage applied so that a greater ampere-hour flow of current was required to produce a given rise of resistance in the case of high-voltage specimens than where the voltage was low. The results of the tests for anode effects on low voltage show an average increase of resistance to 137 times the original value at the end of 5500 hr. The results of the tests for cathode effects on all voltages show an average increase to 14 times the original resistance at the end of about the same period. The anode tests on high voltage do not show such a large increase of resistance as on low voltage; but this is due, no doubt, to the fact that owing to early destruction and short duration of tests.

A rise of resistance of sufficient magnitude to cause the reduction of current through a specimen of reinforced concrete to less than 1% of its original value was recognized as a phenomenon of great importance and was therefore given special consideration.

CAUSE AND LOCATION OF THE RISE OF RESISTANCE—By measuring potential drops between various points between the anode and cathode, care being taken to eliminate effects of polarization at the electrodes, it was found that most of the rise of resistance was at the surface of the concrete nearest the cathode and is due to a film which forms on, and perhaps to some extent within the surface of the concrete. Analysis shows that there is a large accumulation of calcium carbonate at this point and the cause of the rise of resistance is evidently due to the concentration of $Ca(OH)_2$ near the cathode surface by the current, where it comes in contact with CO_2 absorbed by the water from the air with the resulting precipitation of $CaCO_3$ within the pores of the concrete near the surface of the specimen thus plugging up these pores, and forming a nearly impermeable wall.

In case the imbedded electrode is made the cathode, the rise of resistance, while still greatly in evidence, is much less marked than when the imbedded electrode is anode. This might be expected from the fact that CO_2 has very little access to the $Ca(OH)_2$ at the surface of the imbedded electrode, and hence the tendency to produce a dense precipitate of $CaCO_3$ would be very slow. In fact, there is good reason to believe that the rise of resistance which takes place at the imbedded cathode is due chiefly to the liberation of gases at that point by the electrolysis of water. Variations of the resistance of bubbling specimens is as much as 10 to 20% while current is flowing continuously; the resistance drops back quite close to its original value when the current is discontinued for a few days and the gas permitted to diffuse.

EFFECT ON RISE OF RESISTANCE BY ADDING SALTS TO CONCRETE—A not uncommon practice in connection with the placing of concrete in cold weather is to dissolve 2 or 3% of salt (NaCl), or calcium chloride ($CaCl_2$), in the water which is used in making the mortar. It is not definitely known that this has any detrimental effect upon the concrete itself, but it is of material benefit in lowering the freezing point of the mixture while setting. The effect of this addition of salt to concrete in making, upon its electrical properties, seemed to invite investigation, so several tests were made in the usual manner with salt added in various ways. In all cases where such specimens were tested as anodes the damage was not only found to be very much hastened but instead of the usual increase of resistance with passage of current, the resistance actually decreased in some cases as the test proceeded. Two specimens in particular which were made up with no addition of salt, but which were tested in a 3% salt solution, decreased from 115 ohms to 80 ohms and 130 ohms to 60 ohms at the end of 410 hr. and 1490 hr. respectively.

A second test, in which the concrete was made up with a 10% salt solution and tested as anode on 15 volts over a period of five days, showed a decrease of resistance of 25%. Cathode specimens tested in a manner similar to the tests on the first two specimens mentioned showed an increase of resistance of 20 to 25% in the same length of time. This marked difference in character from that of all the other specimens tested in this work must be attributed chiefly to the salt, or, more broadly, to the presence of an excessive amount of the acid radical, chlorine. The currents taken by the specimens were rather high in most cases, and hence heating may have had something to do with the decrease of resistance instead of an increase of resistance with time, but it can not account for all of it. In tests on specimens to which no salt was added the temperature rise was nearly as great but the rise of resistance never failed to occur.

The chief factor is the presence of sodium chloride in a solution preventing the precipitation of calcium carbonate at ordinary temperatures, so that plugging of the pores cannot occur. This phenomenon is, therefore, a corroboration

of the explanation above given for the rise of resistance of specimens of normal concrete.

RISE OF RESISTANCE IN DAMP EARTH—Laboratory conditions have provided a liquid electrolyte in all cases. In practice, wet soil would more often be the electrolyte if conditions were favorable to electrolysis. Current would also be likely to flow between sections of reinforcement without leaving the concrete. In order to determine whether or not this same rise of resistance would occur in the case of concrete embedded in soil, two blocks of concrete containing embedded iron were molded in holes dug in the earth so that about 3 in. of the block projected above the surface. The blocks were of 1:2½:4 concrete, 3½ ft. square by 2½ ft. deep. Four electrodes were embedded perpendicularly to a depth of 2 ft. in each block with 4 or 5 in. of each one projecting for electrical contact purposes. These electrodes consisted of 1-in. iron rods, 1-in. iron pipe and a length of 4-in. wrought-iron pipe. The blocks were placed about 2 ft. apart. Three of the eight electrodes were connected to the negative terminal of the 15-volt circuit and the remaining five to the positive terminal. Current flowed continuously except for about 33 hr. each week. Part of the current which flowed to the positive electrodes flowed out through the surface of the blocks to the soil and to a large ground plate near-by, which was connected to the negative terminal of the 15-volt circuit. The rest returned by way of the electrodes which were connected to the negative terminal of the 15-volt circuit. Current readings were taken on each electrode from time to time. The values of the current on each electrode were plotted as a function of time and showed that the rise of resistance takes place but it is a longer time in appearing than in the case of specimens tested in jars.

VARIATIONS IN MAGNITUDE OF CORROSION OF IRON—It is well known that in cases of corrosion by electric currents, the total amount of corrosion does not in general correspond with Faraday's law, but is in many cases considerably less than the theoretical amount. In particular, in the case of concrete, it was to be expected that the actual amount of corrosion would be less than that expected from Faraday's law, because of the alkalinity of the electrolyte. This obviously has an important bearing on the subject of electrolysis in reinforced concrete, because the extent of the reduction in the amount of corrosion determines the damage which will result to the reinforcing material by a given ampere discharge, and also determines in greater or less degree the extent of the injury to the concrete itself.

A number of tests were carried out using different cements, and every effort was made to have the conditions of the tests as nearly uniform as possible. The specimens consisted of 2-in. cubes of 1:2 mortar made up wet with standard Ottawa sand and distilled water. A round iron electrode ¼ in. in diameter by 2¼ in. long was embedded 1¼ in. deep in each cube. Before embedding these electrodes they were filed to a bright surface and a short, bare copper wire soldered to each one. This was followed by weighing them on a balance having a sensibility of 0.0005 gram, the average weight of the anodes being about 14 grams. In order to prevent natural corrosion of the exposed portion of the electrode while the specimen was immersed in distilled water undergoing the setting process, it was painted with two coats of an alkali-resisting preservative paint. After two weeks' setting, each specimen was placed in a crystallizing dish and surrounded by a sheet-iron cylinder which served as the cathode. The dish was then filled to about ¼ in. from the top of the cube with distilled water, and the specimen put in circuit on 15 volts with the central electrode anode. The current density averaged about 0.025 amp. per sq.in. of anode surface at the beginning and gradually decreased to about 0.005 amp. per sq.in. at the end. Current flowed continuously with the exception of 33 hr. each week, current readings being taken at intervals which increased as the test proceeded. When cracking occurred, or, if cracking did not occur, as soon as a sufficient number of ampere-hours had passed to give a reliable indication of the efficiency of corrosion, the specimens were removed from the circuit and broken open. The corroded electrodes were cleaned by an electrolytic process and then weighed and the losses determined. The test of each cement was made on a set of three cubes, one being kept without current as a check on natural corrosion and the other two tested electrically as described.

The check specimens showed very little natural corrosion, the losses in these cases being hardly greater than the errors which would be expected to occur in weighing. The "efficiencies of corrosion" show beyond a doubt that under the conditions of these experiments but a small fraction of the current passed was effective in dissolving iron. The variations in efficiency of corrosion between different cements are quite marked; the chemical analyses of these cements show that there is apparently no relation between the variations in the efficiency of corrosion and the SO_3 content of a cement. This is not surprising, however, when the fact is taken into account that in every case the mortar contains more than enough $CaSO_4$ to saturate the absorbed water. The variations in efficiencies of corrosion are, therefore, quite unaccountable on that basis.

One of the most important facts noted was the great drop in efficiency of corrosion attendant upon a decrease of the voltage through a wide range of values and the consequent accompanying decrease of current density at the surface of the electrode. Two high-voltage-test specimens were in a condition such as to permit cleaning and weighing to determine the loss by electrolysis. One showed an efficiency of corrosion of 37% and the other of 40.8%. From one extremely high value of 10% for a 15-volt-test specimen the efficiencies of corrosion for this group varied downward, decreasing nearly to zero in many cases. For the type of specimen used in these tests and for the conditions of the tests it seems that corrosion practically ceases at a voltage somewhat below 15 volts.

Increasing the current density in the ratio of 1 to 40 caused no material change in the efficiency of corrosion for any given temperature. Below 50° the efficiency of corrosion is very small, but above that temperature rises rapidly. Since current density has been shown to have no direct effect on efficiency of corrosion the great difference in efficiency of corrosion between high- and low-voltage-test specimens must therefore be atttributed to the heating effect of the current. If the heating effect of the current is sufficient to raise the internal temperatures of the specimens to 50° C. or more, active corrosion begins. Below 50° the iron remains passive, or nearly so. This has been found to be strictly true, however, only for normal concrete specimens to which no foreign ingredient of marked corrosive tendencies has been added as will presently appear.

EFFECT OF SALT ON EFFICIENCY OF CORROSION—The results of low-voltage tests on concrete specimens to which a small percentage of salt (NaCl) had been added showed that passivity of iron in concrete is thereby destroyed almost completely for all voltage and conditions of temperature. Calcium chloride has the same effect, the chlorine ion being the one which affects the iron. In one case it was found that the addition of 0.02% by weight of the cement of $CaCl_2$ was sufficient to cause a very marked increase in the efficiency of corrosion, while 0.33% caused the efficiency of corrosion to rise to 80%.

EFFECT OF AGING CONCRETE ON EFFICIENCY OF CORROSION—Since passivity of iron in normal concrete is brought about largely by the presence of calcium hydroxide in solution in the pores of the concrete, the question arises as to whether the low values of the efficiency of corrosion obtained with low voltages in comparatively new concrete would still be obtained after the concrete is several years old. There is a possibility that after the $Ca(OH)_2$ has become thoroughly carbonated the passivity of iron may be destroyed for all voltages and

temperature conditions. Tests are in progress relating to this phase of the matter, but no results are available at the present time.

Possibilities of Trouble from Electrolysis in Concrete Structures Under Practical Conditions

In connection with these investigations, every effort has been made to study the matter from the practical standpoint also, in order to determine, as far as possible, to what extent the conditions under which concrete can be injured in the laboratory, may be expected to obtain in practice.

CONDITIONS NECESSARY FOR DAMAGE TO OCCUR—It has been seen that the most important essentials in the production of unsound concrete by electrolysis are moisture and a difference of potential between electrodes in contact with the mass of the concrete.

The condition that the electric current must flow between electrodes in contact with the concrete, cannot be too strongly emphasized. The conduction being electrolytic, the reactions take place only at the electrodes. In the absence of such electrodes no reactions occur within the concrete; the only effect, in such case, would be the slow removal of the water soluble constituents and the result would not be essentially different from that of slow water seepage.

SOURCES OF STRAY CURRENTS—If there be electrodes imbedded within the concrete, as in the case of reinforced-concrete structures, the electrode effects described may be expected with sufficient voltage. The sources of potential differences in concrete structures may be classed under two heads: (1) those due to direct contact between the conductors of lighting or power circuits, and some parts of the building, and (2) those having origin in stray railway currents, etc. The former may happen in any building containing electric wires, through defective insulation. It is not necessary, of course, that both sides of the line be grounded in the building itself since, if one side is grounded on the building and the other remotely, those portions of the building itself near the grounded wire will be subjected to a considerable difference of potential. If the wire be grounded directly on the concrete, and not on reinforcement, the small cross-section of the current path near the point of contact will localize most of the total drop of potential to a restricted region near the wire; it is only here that any damage may be expected, and since the current will be small the damage, if any, will be small. Ultimately the current would pass out into the earth through the footings and foundations, and through pipe systems entering the building. As a rule the cross-section of these paths is so large in the aggregate, that the potential gradients would not be sufficient to raise the temperature appreciably and no appreciable damage would occur.

If the current flows to the building from outside there would, in time, be some softening of the concrete in a thin layer under and around the steel structure terminating in the footing, but this would be under compression and not subjected to shear, so that failure here is extremely improbable. The only places where trouble is to be expected due to grounding of power wires directly on the concrete inside of a building, is in the region close to the point of ground.

If the power wire be grounded directly on reinforcing material, the condition will be more serious; the extent of the danger will be greater if there is a large quantity of the reinforcing material in metallic contact with the electric circuit. If this comprises a large part of the total reinforcement of the building the condition might be serious, irrespective of whether the positive or negative side of the line is grounded. If the ground is on the positive side the potential gradient near the reinforcement may become high enough to cause rapid corrosion and consequent destruction of the reinforcing material. If, on the other hand, the reinforcing material be negative, the concrete near the surface of the iron would be softened and practically lose its bond; this would probably be the more serious condition of the two, since it might not become known until a large portion of the building had become weakened. However, this is a trouble that can be readily guarded against.

The other course of possible trouble is the railway ground return. The current may enter a building in two different ways. If the foundations under the opposite sides are at different potentials there would be a tendency for a certain amount of current to flow up through the foundation on one side, through the walls and floors, and out through the foundation on the other side. This condition need not cause alarm. In the course of numerous electrolysis surveys in various cities, we have found that such a potential difference exceeding a few volts is extremely rare, and would be distributed over such distance that the potential gradient would not be sufficient to cause appreciable trouble, in view of our experimental results that a potential gradient of at least several volts per inch is required before damage results.

Stray currents may enter a building through water or gas pipes, lead cable sheaths, etc., and larger differences of potential may be brought between different portions of the building and between the building and the earth. If the pipe systems come in contact only with concrete and not with reinforcing material, any slight damage would be confined chiefly to the immediate vicinity of the pipes or cables. But if the pipes come into metallic contact with the reinforcing material, differences of potential of serious magnitude may be produced in this way, some instances having been brought to our attention in which the reinforcing material was from 5 to 15 volts above or below the earth. Under most conditions damage would proceed very slowly under such differences of potential as these, but nevertheless wherever voltages of this magnitude exist it should be regarded as a dangerous condition and should be remedied at once.

DANGER DUE TO PRESENCE OF SALT—The above statements in regard to the liability of damage under low or moderate differences of potential apply only to concrete which contains no appreciable quantities of salt. If salt has been added to the concrete during construction, or if it comes into contact with salt water afterward, much greater precautions are necessary in order to prevent damage.

We have examined a considerable number of cases in which damage to concrete structures has been attributed to electric currents. Some of these have been reinforced structures and some have been without reinforcement. Among these we have not found any non-reinforced structures in which the conditions indicated that electric currents could in any way be responsible for the damage. Among the reinforced structures which have been called to our attention there are some in which electrolysis has been at least a contributing cause of the damage. We have not, however, seen any case in which serious damage has occurred in which there was not also present a considerable quantity of salt in the concrete, either from having been put there during construction, or from contact with salt water in service.

SPECIFIC CASES OF TROUBLE—In an examination, condition of concrete was carefully compared with that of specimens known to have been injured by current. Potential measurements were made between different parts of the structure, and between it and surrounding structures, although the resistance between different structures is usually unknown, and the voltages gave no idea of the current flow. By comparing the potential gradients observed with those used in the laboratory and considering also the age of the structure and any special conditions which may have influenced it, we were enabled to judge as to whether it was possible for the damage to have been caused by electric currents.

One or two specific cases may be mentioned. The concrete foundations of a small bridge across the Gowanus Canal, South Brooklyn, New York City, developed large cracks in a number of places; on first reports we were inclined to attribute the trouble to electrolysis, since a trolley line passed over the bridge. On examination, however, it was found that where failure occurred there was no reinforcement. There was no similarity between the damage found here and any of the laboratory effects. Po-

tential measurements showed that a maximum potential difference of 1.6 volts, and potential gradients, between points 2 ft. apart near some of the cracks, ranging from 0.01 to 0.05 volts per foot. Further, there was evidence that the foundations of the structure were unstable as shown by the fact that they had moved several inches toward the canal since they were built.

Certain concrete-lined railway tunnels had suffered considerable damage due to disintegration of the concrete, and in some instances within a few months after work had been completed. A voltage survey throughout the tunnels was made, potential measurements being taken between points on the tunnel walls in both a vertical and horizontal direction, and in some cases holes were drilled in the wall and voltage measurements made in a direction at right angles to the surface. Some measurements were also made between walls on opposite sides of the tunnel, and in two or three cases between the tunnel wall and the rails. Measurements were classified as: (1) those where the concrete was very badly disintegrated, (2) those where there was but slight disintegration, and (3) those where the concrete was perfectly sound. A comparison of the magnitudes of the voltages showed first that there was no definite relationship between the magnitude of the potential gradients observed and the physical condition of the concrete. The average voltage per foot in the first group where the concrete was very badly damaged was 0.017. In the second group, there was an average of 0.011 volt per foot, while in the third group the gradient was 0.027 volt per foot. These figures do not support the view that the trouble is caused by stray electric currents. It should be noted that in a number of places measurements were obtained where the concrete was very wet and yet in perfect condition, and in which the potential gradients were of the same order as those in the first group where the concrete was badly damaged.

The fact that there is no definite relationship between the voltages observed and the condition of the concrete is not surprising, particularly in view of the low values of most of the readings. In our experiments at the Bureau of Standards we have subjected blocks of wet concrete to potential gradients varying from very low values up to several thousand times the average values observed in these tunnels, for a period of 1½ years, without producing any appreciable diminution in the crushing strength of the main body of the concrete, and with no evidence of any other action except in the immediate vicinity of the electrodes. We wish to emphasize the fact that in concrete, as in all other electrolytic conductors, all chemical reactions which take place as a result of the current are essentially electrode effects, and they do not, except by progressive action from the electrodes, affect the mass of the concrete.

In the region of the cathode, there is a softening of the concrete, but as pointed out above, it begins as a thin film on the surface of the cathode and very slowly progresses outward. We have found that the rate at which the softened region moves outward from the cathode, with gradients up to 5000 times those in the tunnels, is not over about an inch per year. The theory that damage of this nature could have spread over such vast areas in so short a time is untenable. After a careful study of the conditions existing in these tunnels, we were forced to the conclusion that electrolysis has played no appreciable part in the damage which has been experienced.

VOLTAGE MEASUREMENTS ON CONCRETE STRUCTURES—It is not alone important to be able to read the voltages correctly, but it is equally important to take the measurements between the proper points and properly to interpret them. We have known cases where damage has been attributed to stray currents on the strength of potential measurements taken between the structure and the rails of a near-by steam railroad. The rails near the structure, and for a half mile or so on either side were supported out of contact with the soil on wooden ties so that the potential measurement that was being taken was between the structure and a point several thousand feet distant. It seems almost needless to say that a potential measurement of this kind is worse than worthless as an indication of the electrical condition of the structure under examination. If we are to get an idea of the extent to which currents may be flowing in a building by means of potential measurements it is necessary that we take the potential readings between two points within or on the building itself and not between the building and some other structure more or less insulated from it. Similarly if we wish to determine whether current is flowing into or out of a building we should make the potential measurement between points on the building and thoroughly grounded points near by. Failure to keep these facts in view appears to have been responsible for a good many eroneous reports of damage to concrete structures by stray currents.

Protective Measures

EXCLUSION OF SALT—In considering measures to prevent possible damage, it is not necessary to consider structures which have no metal imbedded in them, since these are immune. As regards reinforced-concrete buildings (or concrete buildings with metal conduits imbedded in the concrete) we have to consider: (1) structures building, and (2) structures completed.

In the first case salt should be omitted altogether, if there is the slightest probability of action by electric current. Calcium chloride is quite as bad. Since the addition of even a fraction of 1% chlorine is sufficient to increase the rate of damage a hundred fold, it is impossible to use a sufficient quantity of salt to lower the freezing point of concrete perceptibly without making the structure decidedly more vulnerable to the action of electric currents.

WATERPROOFING BELOW GRADE — A good deal can be accomplished also by proper construction of the basement, foundations and footings of the building. If beneath the foundations and outside the basement walls a layer of insulating material be placed it will prevent the access of electric currents through the building foundations. While such insulating materials can readily be made that will give a permanent increase of resistance, we have yet to see one that will, in the presence of water, remain completely insulating over a period of years. A waterproofing medium, such, for instance, as one made up of multiple layers of fabric treated with pitch will have sufficient insulating value to be quite effective in preventing the passage of electric currents, and would probably be of considerable value even after the waterproofing has ceased to be perfect.

ADDITION OF WATERPROOF COMPOUNDS —Attempts to increase the resistance of concrete by the addition of so called waterproofing compounds to the cement before mixing have uniformly failed to give satisfactory results. None of these compounds or mixtures that have been brought to our attention are of great value in this connection, and reliance should not be placed on them.

CONSTRUCTION OF FOUNDATIONS — A good deal can be done in the way of increasing the resistance between the building and ground by a proper selection of materials for the foundations. Blocks of granite, frequently used in such work, have a much higher resistance than concrete, a number of specimens tested by us showing, in water-soaked condition, a resistance approximately 100 times as great as that of wet concrete. If granite blocks be interposed between the footings and the soil the tendency of the building to pick up stray currents will be greatly reduced. We regard this as of secondary importance if the potential differences in the region of the building are as small as usual, since any currents picked up will be too small to do appreciable harm. In some cases, however, the use of granite footings may be justified as a measure of precaution.

The foregoing statements are not applicable to structures already built, but the following preventive measured may, as a rule, be applied to old as well as new structures.

ELECTRIC WIRING—In wiring a concrete building with direct-current circuits it is of the greatest importance that the con-

struction be such as to preclude the possibility of either side of the circuit coming in contact with the concrete. Insulation should be of the best and wires should be in continuous metal conduit. Periodic tests of insulation should be made and defects remedied. If the power supply is a private plant belonging to the building the installation of ground detectors is desirable. With these precautions it is not necessary to insulate conduits from buildings.

INSULATION OF PIPES AND CABLES—By far the most important path of entry of electric currents into concrete structures from outside sources is through the pipes and lead-cable sheaths, etc., which enter the building; it is only through these paths, as a rule, that sufficiently high potential differences can be produced to result in damage to the structure. The most effective remedy is to introduce insulating joints into the pipes before they enter the building. Such joints are now used in many places, although, not so far as we are aware, in this particular connection. The only condition to guard against is the possibility of developing too high a difference of potential across the joint which might give rise to electrolytic injury to the pipes; but this can be obviated by proper construction of the joint, as by giving it a long leakage path and by putting several insulating joints in series.

To use them on one side only, when a pipe line passes through a building, would result in making the building more strongly positive or negative to the earth than without such joints. If the insulating joints are put on both sides, however, there will be no likelihood of currents entering the structure to any appreciable extent, except in special cases where the difference of potential between the broken sections of pipe become very high, say of the order of 10 or 15 volts or more. Then trouble can be alleviated by shunting around the insulated ends of pipe within the building by means of a copper cable, preferably insulated.

There are more serious objections to inserting insulating joints in leaded cables than in the case of water or gas pipes. It would usually be better to insulate the cable from the building, by carrying it on wooden supports.

DANGER OF MAKING REINFORCEMENT NEGATIVE—A protection which has frequently been proposed, making the reinforcement continuous throughout and connecting it to the negative terminal of a low-voltage generator, should be referred to here, but only to point out the danger. Corrosion of reinforcement might be prevented but there is greater danger on account of destruction of bond.

GROUNDING METALLIC CONDUITS—The practice, at present followed in many instances and made compulsory in some city ordinances, of grounding all metallic conduits in contact with the concrete, is not to be recommended as a general rule. In the case of metallic conduit imbedded in concrete we would advise against grounding. If the ground were on the water pipes, as is common, and if the district were one in which the pipes were at a higher or lower potential than the earth, the conduits would be made either anode or cathode. This condition would be worse than the unrestricted entry of pipe systems into the buildings, which should be guarded against as pointed out above. Connection to a ground plate would bring the conduit system to the same potential as the ground plate. The footings of the building remote from the ground plate may show a considerable difference of potential against the ground plate and consequently against the conduit, thus giving rise to flow of current between the conduit and the structure itself.

IMPROVING RAILWAY NEGATIVE RETURNS—The most effective way of reducing damage from stray currents is by removing, as far as practicable, the cause of such currents. This means proper design, construction and maintenance of the railway distribution system. The protective measures mentioned above are to be regarded as precautionary ones, rendered necessary only by failure of railway companies to provide adequate path for return currents. If the negative return systems of the electric railways of this country were uniformly as good as those maintained in the principal countries abroad, the problem of damage to buildings by stray currents would practically disappear.

The Development of Large Turbine Gears for Ships

Readers of ENGINEERING NEWS have been made familiar with Sir Charles Parsons' use of cut gears in rigid bearings for ship propulsion by steam turbines (see ENGINEERING NEWS, Apr. 14, 1910, p. 426; Apr. 21, 1910, p. 466; June 6, 1912, p. 1081; May 25, 1911, p. 638) and with the gears in floating frames brought out by the late Admiral G. W. Melville and J. H. Macalpine under the direction of Geo. Westinghouse (see ENGINEERING NEWS, Oct. 29, Nov. 11, Dec. 9, Dec. 30, 1909, pp. 442, 519, 646, 724, 730; Apr. 7, 1911, p. 412; Nov. 16, 1911, p. 591). It will be recalled that the first Melville-Macalpine gear frames maintained alignment and uniform tooth pressures through the slight movements of steel members of the pinion supports, but that this was afterward replaced by an arrangement of hydraulic cylinders and pistons supporting the pinion bearings. Detailed reasons for the change were not then given, but interesting information is now given, on the whole development of these devices, by Mr. Westinghouse in a communication to the London *Times'* Engineering Supplement, Nov. 20, 1912. The occasion of the letter was an article by Sir William White in that paper of Oct. 9, Mr. Westinghouse evidently feeling that the limitations of a rigid-supported gear had not been sufficiently outlined or the necessity of devices for automatic alignment and equilization of tooth pressure. A number of extracts from the letter follow.

MR. WESTINGHOUSE'S LETTER

GENERAL DEVELOPMENTS AND COMPARISONS—We evidently differ as to who initiated the application of reducing gears to modern marine engineering and as to the respective importance of the work done by Sir Charles Parsons and myself in connection therewith.

Sir William's article completely confirms the conclusion reached by Admiral G. W. Melville and J. H. Macalpine in their report to me in May, 1904, made after a prolonged investigation of the questions involved, in which they said:

> If one could devise a means of reconciling, in a practical manner, the necessary high speed of revolution of the turbine with the comparatively low rate of revolution required by an efficient propeller, the problem would be solved and the turbine would practically wipe out the reciprocating engine for the propulsion of ships. The solution of this problem would be a stroke of great genius.

Admiral Melville, after presenting the report, argued for speed-reducing gearing and finally agreed with me that something new was required to meet the difficulty of first establishing and then maintaining a perfect tooth contact of uniform pressure in gearing having fixed bearings and the length of teeth needed for the large powers demanded in the equipment of modern naval and merchant vessels.

In 1906 Admiral Melville and Mr. Macalpine came to me with drawings of the floating frame arrangement which they had evolved. As the design seemed to meet the necessities I immediately agreed to undertake the construction of the gearing, which was finally completed and tested in 1909. Patents were granted in several countries for the invention, British Patent No. 18,759 of 1907 having been opened to public inspection on Feb. 23, 1908.

This patent was available to Sir Charles Parsons months before he delivered his James Watt Anniversary Lecture at Greenock, Scotland, Jan. 15, 1909, in which he said:

> We might naturally speculate as to the future, and inquire if there is a possibility of the turbine being constructed to run more slowly and without loss of economy, or whether the propeller can be modified to allow of higher speed of revolution. Or, again, may the solution be found in reverting to some description of gearing—not the primitive wooden spur gearing of half a century ago, but to steel gearing cut by modern machinery with extreme accuracy and running

in an oil bath, helical tooth gearing, or chain gearing, or again some form of electrical or hydraulic gearing? These are questions which are receiving attention in some quarters at the present time, and if a satisfactory solution can be found, then the field of the turbine at sea will be further extended.

This clearly indicates that he had not yet reached any conclusion in the matter of the use of reducing gears; and I am sure Sir William White will agree that the experiment made by Sir Charles Parsons in 1897 with a 10-hp. geared turbine has no more to do with the questions under discussion than the fact that I made and operated a jet steam turbine in 1870 has to do with the subsequent work of others.

I am safe in saying that prior to my bringing out the Melville and Macalpine reducing gear with its self-aligning floating frame, engineers generally had but little faith in the use for marine work of such gearing as was then known.

It must not be overlooked that there had been little or no experience with large gears moving so rapidly as those of Melville and Macalpine, so that when a pitting of the teeth near the pitch line developed (a difficulty mentioned by Sir Charles Parsons in describing the operation of the Vespasian's gears) the cause and a cure had to be worked out to insure durability. A careful study of this difficulty satisfied me that the cause of the pitting was due to the teeth coming together at a velocity of 100 ft. per sec. with a hammer effect. The cure was to substitute a pinion frame hydraulically and elastically aligned in place of the original floating frame.

In my judgment, one familiar with the simplicity and effectiveness of this hydraulically adjusted pinion frame would no more think of resorting to an arrangement of gearing in fixed bearings than of attempting the impossible arrangement of rigidly fixed bearings for a locomotive running at high speed upon an uneven surface.

It is obvious that rigidly carried gears must be cut and dressed with great accuracy before they are put in service, and the scraping of the teeth is a tedious and expensive operation. The pinions and gears of the hydraulically operated system can be inserted in their bearings just as they come from the cutting tool, and in this condition will operate satisfactorily and without objectionable noise.

TOOTH PRESSURES — The admirable operation of the hydraulic floating frame has established the fact that in gears of 1000 to 2000 hp. a maximum tooth pressure of 1000 lb. per lin.in. of gear width is a safe working pressure, for it must be borne in mind that this 1000 lb. is divided practically between two teeth, making the maximum pressure per inch of tooth about 500 lb. If this be so, no great difficulty could have arisen due to excessive tooth pressure on the "Vespasian" even if the axes of the two gears were not exactly in line, because if even only one-quarter of the total length of teeth were to be engaged, there would not result an abrading pressure, and this would also be the case with respect to the gears installed upon the "Normannia" and other ships fitted in a like manner. Stated in practical terms, the width of main gears and length of pinions in the "Vespasian" and "Normannia" are more than double the requirements when absolute uniform tooth pressure is automatically assured. It would, therefore, seem that the Parsons reducing gear with rigid bearings has not yet undergone the test of the high tooth pressure at high speeds which must eventually be provided for, and that Sir Charles Parsons' success has been in avoiding such pressures by making heavy and costly apparatus rather than in producing the lighter gearing rendered possible by the hydraulically aligned pinion frame.

Realizing, in the interest of marine work, the high importance of quickly and conclusively establishing the practicability of the floating frame as well as the probable life of high-speed high-power pinions, I have had constructed, since 1910, 20 sets of reducing gears with an aggregate capacity of 29,000 hp. used for driving electric generators and pumping apparatus. There are now under test two gears of 6000 hp. each, and a number of other sets are under construction, making an aggregate of about 50,000 hp. Some of these gears have run with 100% overload for long periods and others have had excessively severe and rapid fluctuations in load from a few per cent. of full load to over 100% overload. The service of some of these gears measured by speed of the teeth and the work performed, with a negligible amount of wear, if employed in marine work would drive a vessel like the "Vespasian" over 200,000 nautical miles, which has satisfied my engineers that the life of high-speed gearing with the hydraulic floating frame will be much greater than the life of any ship to which these gearings may be fitted.

COMPARISONS FROM PRACTICE — Sir William refers to the two cross-Channel steamers in the service of the London and South-Western Ry. fitted with Sir Charles Parsons' latest turbines and gears, and states that this is "one of the most important and most recent applications of the geared system." I am thus afforded an opportunity to make comparisons based upon these particulars which will bear out the claims I make as to the imperative need in large powers for the use of a floating frame which will not only maintain an even tooth pressure in order to effect a great reduction in the weight and cost of the gear apparatus for the operation of vessels, but which will also reduce vibrations and consequent noise to a minimum.

The installations in the "Normannia" and her sister ship comprise two sets of compound turbines of the usual type, each having a high- and a low-pressure rotor, each turbine operating a shaft with pinions engaging in the main gear connected to the propeller shaft. There are, therefore, four pinion shafts to carry the 6000-hp. maximum developed on the trials; that is to say, 1500 hp. per pinion shaft.

In the "Vespasian," first tried in 1910, the maximum power developed did not exceed 1000 hp. In this case there were also a high- and a low-pressure turbine each driving a pinion, so that the maximum work done per pinion was only 500 hp.

In the case of the U. S. collier "Neptune," with reduction gears fitted with my own hydraulically operated floating frame, there are only two turbines and two pinions, each of which developed a maximum of 4000 hp. on trial. The work done per pinion on the "Neptune" was, therefore, about 2⅔ times the greatest work done per pinion on the "Normannia," which is the largest Parsons installation of which any account has been given.

In the test of the Melville and Macalpine trial apparatus by the Westinghouse Machine Co., in November, 1909, a continuous 40-hr. run was made at 6000 hp., while a maximum of 6800 hp. was obtained, making the work of the pinion more than four times that of the greatest work done by the Parsons pinion on the "Normannia."

In regard to the gears on the "Neptune," Lieut. W. W. Smith, U. S. N., the chief engineer of that vessel, says in a paper published in the *Journal* of the American Society of Naval Engineers for August, 1912:

> In the writer's opinion, the floating frame or similar device for aligning the pinion automatically is absolutely essential for large gears. Ship's mechanics could not be relied upon to make the extremely accurate alignment that would otherwise be required. For military reasons it is of great importance to be able quickly to overhaul and assemble with the absolute assurance that the machine will run perfectly and be absolutely dependable. This feature of the floating frame is of utmost importance in naval work, where self-maintenance and reliability are essential. With the floating frame it is only necessary to adjust the struts according to the plug gages, which any mechanic of ordinary ability can do. The alignment takes care of itself.
>
> There has been practically no wear. The pinion teeth are highly polished and show no signs of cutting. The gear teeth are just barely polished along the pitch line.

I appreciate how disturbing my statements and suggestions have been to Sir William White and Sir Charles Parsons, for both should be able to comprehend the great significance of one of the statements made by me:

> A battleship of the "Utah" or "Florida" class of the United States Navy as now

fitted has eight Parsons turbines requiring together 850,000 blades, which, laid end to end, would reach over 40 miles. With the new type of turbine referred to, with reduction gears, there will be required four turbines having less than 60,000 blades, the aggregate length of which is only two miles.

Interpreted, this means that lighter, more efficient, and much less costly machinery is now available to those desiring to equip vessels than Sir Charles or anyone else has yet presented. It means also such a tremendous reduction in the number and length of blades required for marine work as to insure almost entire freedom from the difficulties which have been so often encountered, not only in marine installations, but in installations for land purposes, facts with which both Sir Charles Parsons and myself have had to become familiar through sad experiences.

It also means not only a great saving in first cost in the building of a battleship, but it also means an increase in its fighting power by making it possible to add from 10 to 20% in the weight of guns, due to the saving in machinery and boiler weights.

I have very high appreciation of the work Sir Charles Parsons has done in the development of marine turbines and an equally high opinion of Sir William White's knowledge of this interesting subject, and I believe we must be in accord in desiring to have brought out all of the advantageous points in connection with the use of geared turbines for marine work.

The Glen Loch Bridge Wreck

A fast passenger train on the main line of the Pennsylvania R.R., was wrecked on Nov. 27, while passing over a girder bridge at Glen Loch, Pa., 25 miles west of Philadelphia. Investigation shows that the cap of a steel column supporting one of the girders had broken. The cap was of defective design, and the wrought-iron cap-plate was placed with its fibers running the wrong way, so that the greatest stress came upon the most brittle section. The plate broke by progressive fracture. Other features of the structure are under criticism.

The remarkable wreck of a fast passenger train on the Pennsylvania R.R., at Glen Loch, Penn., on the late evening of Nov. 27, 1912, while crossing a small plate-girder bridge, was followed by an investigation of cause on the part of a committee of railroad officials appointed by the general manager, S. C. Long. The report of this committee, a very comprehensive document, supplies the essential material for forming a conclusion as to the nature of the accident.

The wreck was a derailment of the train which took place on the first half of the bridge (that is to say, before the front of the train reached the middle of the bridge) and resulted in the train running on the ties, held by the guard rails, to well beyond the bridge, where about half of the train went over the side of the bank, causing the death of four persons. Failure of the cap of a column post,* one of a braced bent supporting the middle of the bridge, was either the cause or the result of the derailment, with the probabilities in favor of the bridge failure having caused the derailment.

*Or other failure of structural parts of the bridge, involving the post-cap fracture as either primary or secondary feature. A full account of the damage to the bridge has not been given.

The failure of the post cap mentioned in the committee's report is of great interest to the structural engineer because the cap was of faulty design and faulty construction, and because the failure is a characteristic instance of "detail fracture." Further, the case carries us back to the period when wrought-iron was the principal material in bridge construction; wrought-iron continued in use on the Pennsylvania R.R. under the late William H. Brown as Chief Engineer for a long time after steel had become the more common bridge material elsewhere.

Circumstances of the Wreck

The circumstances of the wreck may be reviewed very briefly. A long and heavy passenger train on the Pennsylvania main line, westbound from Philadelphia to Cincinnati and points west, train No. 19, drawn by two engines, reached the Glen Loch Bridge across the Trenton Cutoff tracks at 11:40 p.m., Nov. 27, about five minutes behind its schedule, but running at its normal speed, 45 to 50 miles per hour. For some distance back of the wreck it had been drifting on a down-grade, and no steam was used and no brakes were applied during that period; when the engineer first felt that the train was being wrecked, on the bridge, he applied the emergency brake. He describes the first feeling of something wrong thus: "The engine just dropped down and gave a dip and then went up again." It threw him off his seat-box.

The weather was clear and no conditions were abnormal. The train was in good order, and neither at the inspection before leaving Philadelphia nor at the inspections after the wreck was any condition found that revealed any con-

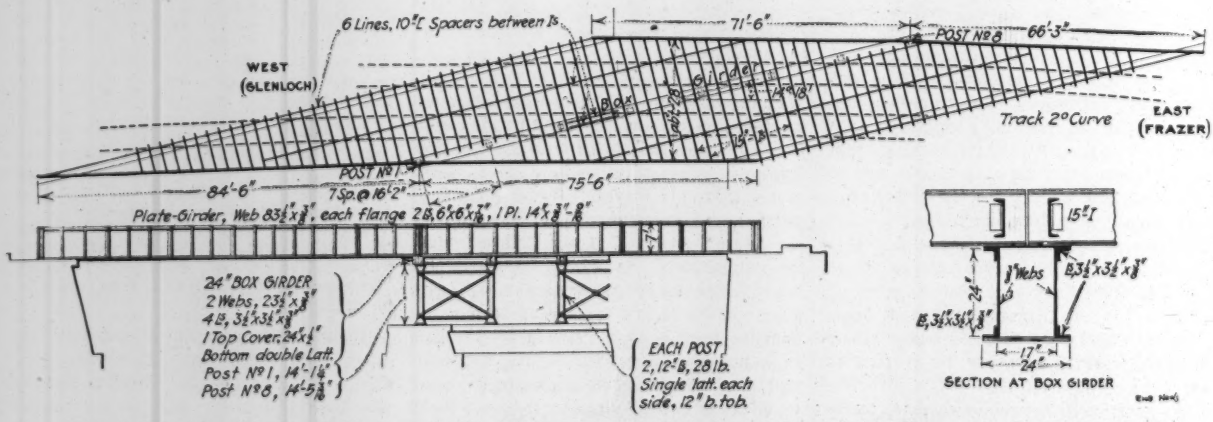
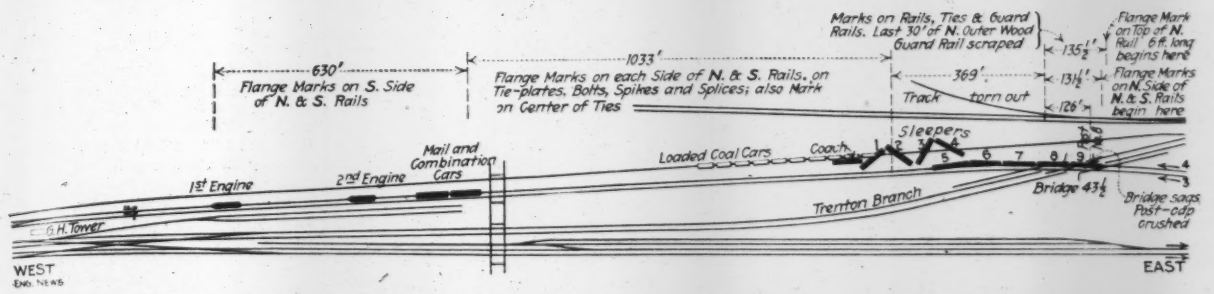


FIG. 1. OUTLINE SKETCH OF STRUCTURAL FRAMING OF BRIDGE 43½, NEAR GLEN LOCH, PENN., ON PENNSYLVANIA R.R. MAIN LINE

(Track carried by cross-ties on wooden stringers, one under each rail. Bridge built in 1891. Material, wrought iron. See Fig. 3 for detail of cap of supporting columns, and sketch of failure of cap.)


Marks on Rails, Ties & Guard Rails. Last 30' of N. Outer Wood Guard Rail scraped
Flange Mark on Top of N. Rail 6 ft. long begins here
630'
1033'
135½'
Flange Marks on S. Side of N. & S. Rails
Flange Marks on each Side of N. & S. Rails, on Tie-plates, Bolts, Spikes and Splices; also Mark on Center of Ties
369'
131½'
Flange Marks on N. Side of N. & S. Rails begin here
Track torn out
126'
Sleepers
Mail and Combination Cars
Loaded Coal Cars
Coach
1st Engine
2nd Engine
Trenton Branch
Bridge 43½
Bridge Seats Post-cap crushed
G.H. Tower
WEST
EAST

(Note—The following marks were recorded in track No. 4 on the bridge, the stationing being counted from the point of the inner guard-rails east of the bridge:

1 + 16.7 Flange mark on top of N. rail from 1 + 16.7 to 1 + 22.8. The rail extends from 1 + 15 to 1 + 48.
1 + 20.5 South iron guard-rail loosened from this point to west end.
1 + 20.5 Flange marks on north side of S. rail start.
1 + 21.3 Flange marks on north side of N. rail start.
1 + 25 Ends of girders; Post No. 8.
1 + 28 to 1 + 30.6 South wooden guard-rail scraped.
1 + 37 Mark on center of tie.
1 + 39.8 Flange marks on north side of S. rail stop.
1 + 40.5 West edge of tie torn off.
1 + 45.6 Flange marks on north side of N. rail stop.
1 + 47.3 Slight indentation on top of N. rail.
1 + 48.9, 1 + 49.6, 1 + 50 } Marks on ties.
1 + 58.2 First flange mark on south side of N. and S. rails.
1 + 76 to 1 + 79 Wooden guard-rail scraped.
1 + 77 West edge of tie torn off.
1 + 97, 2 + 07 } Marks on center of ties.
2 + 13, 2 + 14.5, 2 + 16 } West edges of ties torn off.
2 + 20 to 2 + 51 North guard-timber scraped.
2 + 52 Track torn out 369 ft. beginning here.

FIG. 2 LOCATION PLAN OF GLEN LOCH WRECK, SHOWING DERAILED TRAIN AND EFFECT ON TRACK

tributing cause to a derailment, either in the way of missing parts or defective running gear.

Five to ten minutes before, a freight train had passed over this bridge, also bound west. It was proceeding on the passenger track (No. 4) because another freight was occupying the westbound freight track (No. 3, left-hand or lower track in the plan sketch, Fig. 1); it returned to the freight track at the cross-over west of GH tower, half a mile beyond the bridge. The freight train was heavier than the following passenger train, and like it was a double-header. While it was passing over the bridge the rear end of the freight train on track No. 3 was also on the bridge, just passing off to the westward. Neither of the two trains were found to be in any way out of order, or to have missing parts or defective elements that might have contributed to the derailment. The same is true of the various preceding trains (12 trains passed over the bridge westbound in the 2½ hr. preceding the wreck).

The drawing, Fig. 1 herewith, shows the structural elements of the bridge, from the original construction drawings. Fig. 2 shows the results of the derailment. The drawing, Fig. 3, shows the cap of one of the center-bent posts of the bridge, and the condition of the cap of the northeasterly post (Post No. 8) after the wreck.

Two small items in the testimony suggest that something may have been wrong at the bridge before the passenger train came upon it. They are mentioned in the committee's report, printed below. It will be seen that they can be given but little weight in attempting to reach a conclusion as to whether the post-cap broke under the passenger train or under the preceding freight train.

The report of the Pennsylvania Railroad committee to S. C. Long, General Manager, is as follows:

The undersigned, the committee appointed for the purpose of ascertaining cause of the wreck of train No. 19 at Bridge 43½, Philadelphia Division, on Nov. 27, 1912, beg leave to report as follows:

Train No. 19 left Broad St. station, Philadelphia, at 10:54 p.m., six minutes late, composed of the following engines and cars: Helper engine E-3a 2426; train engine E-3a 1618; one mail car; one combined car; one coach; nine sleepers. It received the usual and proper inspection and left Philadelphia in good condition. It passed Frazer at 11:38 p.m. in proper condition, six minutes late, having made up no lost time on schedule. Train approached Bridge 43½ at a speed of about 48 miles per hour, and was derailed at the bridge about 11:40 p.m.

The first engine broke loose at the bridge and ran a distance of about 2000 ft. with No. 1 driving wheels and No. 4 tender truck wheels off the track.* The second engine also broke loose from the train and ran a distance of about 1800 ft. with No. 1 driving wheels and No. 2 and No. 4 tender truck wheels off the track.* The postal car and the combined car broke loose from the coach and ran 1617 ft. with No. 2 and No. 5 pairs of wheels off the track under the postal car and wheels of both trucks off the track under the combined car. The coach was the first car to leave the roadbed, at a point about 600 ft. from the center of the bridge, rolling down the embankment to the right and turning upside down, resting in that position against a steel coal-car standing on No. 7 track at the foot of the embankment. Six sleeping-cars were derailed and fell down the embankment in more or less irregular positions, the first one striking a steel hopper car loaded with bituminous coal at the foot of the embankment, which contributed very largely to the destruction of the sleeping-car. The seventh sleeping-car had the front truck derailed only; the last two remained on the track.

The weather at the time of the accident was clear; temperature, 34° F.

As a result of the accident, two Pullman employees and two passengers were killed, and 49 persons were injured.

The last train ahead of train No. 19 was on No. 4 track, consisting of Class H-8b engine 1728 and Class H-8sb engine 3413 with 73 empty cars, and passed Frazer at 11:31 p.m., and Glen Loch at 11:35 p.m. This train was running at a speed of about 20 miles per hour. The crew on the lead engine did not notice anything unusual while passing over the track and bridge, but the engineman, fireman and brakeman riding the second engine noticed a jolt or jar, referring to it in their testimony as a low joint. The conductor, flagman and rear brakeman, riding in cabin car on the rear end of this train, stated that there was nothing irregular in the track to attract their attention.

P-39, Class H-6a engine 2365, local freight, consisting of 36 cars, passed Frazer at 11:28 p.m. on track No. 3 and stopped at Glen Loch westbound home signal. The rear end of this train passed over the bridge about the same time that the front end of Extra 3413 passed. The flagman of P-39 got off promptly and went back a distance of eight or nine car lengths and was standing on the Trenton Branch track west of the bridge when No. 19 approached. At the same time P-65 westbound was standing on the Trenton Branch track west of the bridge and flagman Donnelly was back protecting his train almost opposite where Flagman Haines of P-39 was standing, and both of these men watched the approach of train No. 19 and both state that just before No. 19 reached the east end of the bridge they noticed considerable fire flying from under the first engine; immediately after this they heard the sound of the engines rumbling on the bridge.

Several other trains passed this point a short time ahead of those named, but the evidence of enginemen and trainmen would indicate no irregularity of bridge or track.

No trespassers were noticed in that vicinity by any of the witnesses.

*These five pairs of wheels went off the rail to left, or south, i.e., inside of the 2° curve.—Ed.

In a careful examination of the locomotives before any work was done to them after the accident, the only defects found were those incident to having the No. 1 driving wheels on both locomotives and the No. 4 tender truck wheels behind the first locomotive and No. 2 and No. 4 tender truck wheels behind the second locomotive off the track. The coupler bracket and the pilot braces on the front end of the second engine were loosened, and the coupler at the rear end of the tender behind the second engine was entirely missing and could not be found at any place on the track between the bridge and where the engines were standing.

A careful inspection of the mail and combined cars remaining on the roadbed failed to show any defective parts that could in any way have caused the accident.

A careful study of the work reports shows clearly that no defects existed on the locomotives or tenders that could in any way have contributed to their derailment.

Bridge No. 43½ is an iron structure and was constructed by the firm of Cofrode & Saylor in 1890. It carries the two main westbound tracks, Nos. 3 and 4, on a curve of 2° to the left, over the eastbound and westbound tracks of the Trenton Branch, at an angle of approximately 15°. The superelevation of the outer rail is 3 in. The grade breaks on the bridge, ascending toward the east at the rate of 0.29% and descending toward the west at the rate of 0.72%. The bridge is of two spans of half-through built girders, the ends resting upon stone abutments.

The center pier is composed of a stone wall or pier, about 5 ft. high above the level of the Trenton Branch rails, upon which there are eight iron latticed posts 12 in. square, composed of two 12-in. channels joined together with latticed bracing, the tops of the posts being enlarged to 24x24 in. in the form of a cover-plate resting upon the top of the column and supported longitudinally in the direction of the center pier by gusset plates on either side of the column, the top length of the gusset plates being 24 in., the height of the plates being 18 in. and the bottom of the gusset being the width of column (12 in.). In addition, the outer edges of the cover plates are supported by 4x6-in. angles riveted to the same and to the upper edge of the gusset plates and column channels. Upon the top of these eight columns rests a built box girder with web plates 17 in. apart and 24 in. high, these web plates being attached to a top cover plate 24 in. wide by a suitable angle. The bottom edges of the web plates are stiffened by angle irons. Where the girder rests upon the cover plate, at the head of the post, it is held in position by four bolts through the lower angles and the cover plate of the posts. The bottom edges of the web plates are held in position by lattice bracing.

The main girders are supported upon the stone abutments and their abutting ends in the center are held in position on the top of the box girder by two bolts in the end of each of the main girders through the top cover plate of the box girders. The floor of the bridge, supporting the tracks, is composed of I-beams riveted to the main girders above the lower flange and supported at the center pier on the top of the box girder, which rests upon the latticed iron columns. On top of the floor-beams the track structure is composed of wooden stringers, lengthwise under the rails, properly dapped over the floor beams, the cross-ties being laid directly upon the wooden stringers, and the rails spiked to the same.

The standard regulations require all bridges to be inspected monthly by the Bridge Inspector for detail defects, semi-annually by the Division Engineer and Master Carpenter, and annually a special deflection test is required to be made by the Bridge Inspector. Bridge 43½, in accordance with these regulations, was inspected in detail on Nov. 18 by the Bridge Inspector, at which time nothing unusual was observed, every part being reported as "Good" except the back walls of the abutments, which were reported as "Fair," which comment had reference to slight displacement of several stones, due to the wedging effect of the ballast on the "land side," which in no wise affected the integrity or safety of the structure. The semiannual inspection by the Division Engineer and Master Carpenter was made on Oct. 7, which inspection disclosed no imperfection affecting the integrity or safety of the bridge. The annual deflection test was made on Nov. 26, one day before the wreck, and this test developed no conditions that would indicate weakness in the structure.

The two tracks on either side of the bridge are well ballasted with trap rock. The ties, spaced 18 to each 33 ft. of track, are practically all white oak, each with 8 in. or more face, all ties tieplated with shoulder tie-plates, and are in first-class condition, the 1912 renewals having been completed and the track put in good shape for the coming winter. The rail on No. 4 track is P. S. openhearth, 100-lb. section, manufactured by Pennsylvania Steel Co., and was laid new on June 9, 1910. The splices are six-hole Bonzano dependent flange type, manufactured by the Pennsylvania Steel Co. The line and surface of No. 4 track on the bridge and on either side was in first-class condition at the time of the accident.

The track on Bridge 43½ and on either side of it is inspected once daily by the track foreman and twice each day by trackwalker and twice each night by the night trackwalker, the latter being checked by a watchman's clock. The track foreman and both trackwalkers testify that their inspections on the day of the accident revealed no defects, the night trackwalker stating that he had spent about 20 min. examining the bridge and tracks shortly after 10 p.m., or about 1½ hr. prior to the accident.

Immediately after the wreck an inspection disclosed a mark or thread line on top of the north rail, beginning at a point 8.3 ft. east of Post No. 8, then continuing westward for a distance of 6.1 ft., where it left the rail to the north side. Further on and about 4 ft. east of Post No. 8 there were marks on the cross-ties of the flanges of wheels, north of the outer rail and between the inner main rail and the inner guard rail.

The end of the box girder over the top of Post No. 8 was depressed about 18 in., due to the rupture of the cover plate at the top of this post. Beyond this point there were numerous marks of derailed wheels.

A laboratory test was made of the ruptured cover plate. This indicates that the plate would have passed the requirements of strength and elongation of the ordinary bridge specifications, as well as of the specifications of merchant bar iron of the Pennsylvania R.R. The plate was placed in position on the top of the column with the fiber longitudinally in the direction of the bridge.

The design of the structure prevented the disclosure of the rupture of the cover

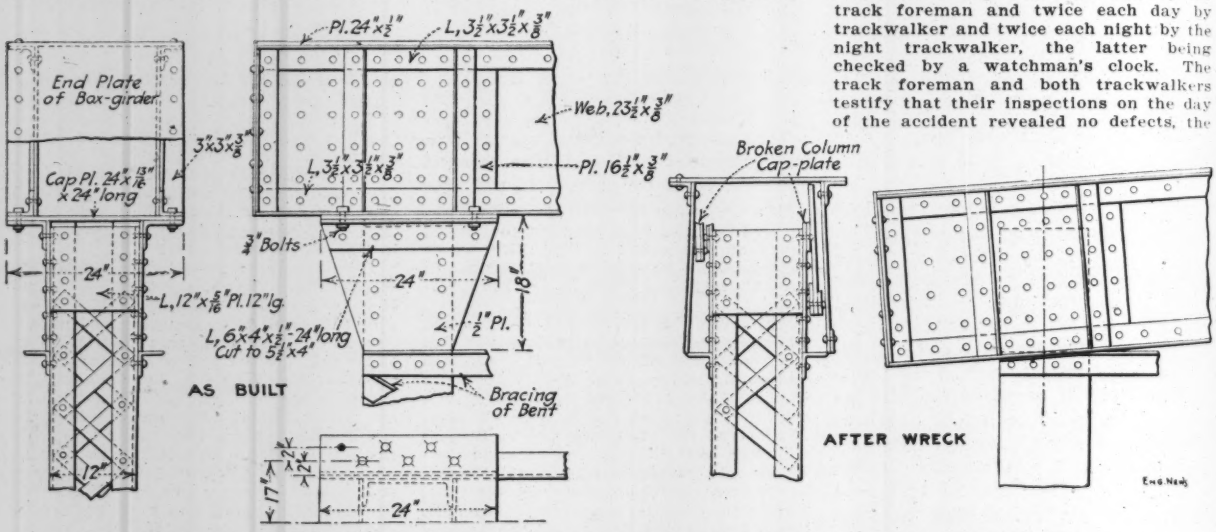


FIG. 3. CAP OF CENTER-BENT POST, BRIDGE 43½, AND SKETCH AFTER FAILURE OF CAP OF POST 8

plate under the usual inspection, except at the front and rear ends of the plate where the accumulation of paint would have a tendency to obscure the same. This detail fracture had no observable effect upon the stability or strength of the structure, as is evidenced by the results of the deflection test made the day before the wreck occurred. This test was made with the bridge under load of passing trains, and did not disclose any weakness in the structure.

After hearing the evidence of those having knowledge of the occurrence and who were responsible for the condition of the track, bridge and equipment, and reviewing the facts above disclosed, our conclusion is that the failure of the bridge was occasioned by the rupture of the cover plate on top of Post No. 8, which permitted the superimposed box girder when loaded to drop down 18 in. This in turn depressed the rails.

shock sufficiently severe to rupture the cover plate of Post No. 8.

(Signed) G. W. Creighton (Chairman),
H. C. Bixler,
J. R. Alexander,
J. P. Yergy,
J. M. Henry,
J. W. Stone.

From one of the appendices to the report we reproduce the wheel-load diagram of one of the passenger-train engines below; the two engines were alike.

The train was 1104 ft. long, and weighed 2,327,780 lb. The heaviest cars in the train, the sleepers, weighed about 150,000 lb. each, loaded.

The locomotives were derailed to the south (driver 1 and tender wheel 4 of first engine, driver 1 and tender wheels 2 and 4 of second engine, the others remaining on the track), while the wrecked cars went off to the north. This must be considered when reading the committee's conclusion that derailment of one of the locomotives (by exterior cause) might have caused the bridge failure. Since the first rail-mark is a very faint mark on top of the north rail, leading off to the north, this can hardly have been made by a locomotive wheel.

Sufficient facts are not presented to furnish definite answers to several questions, e.g.: Did the bridge failure precede and cause the derailment, or *vice versa*? If the former, was the failure caused by the double-headed freight, or by the somewhat lighter but much faster passenger train with its two engines having 31,000-lb. driving-wheel concentrations? If the failure took place under the passenger locomotives, why were these derailed to the inside of the curve?

However, it is clear that in any event the bridge failure is the extraordinary feature of the accident. The committee report does not refer to the behavior of any part of the bridge except the cap of Post 8, though this could not fail and

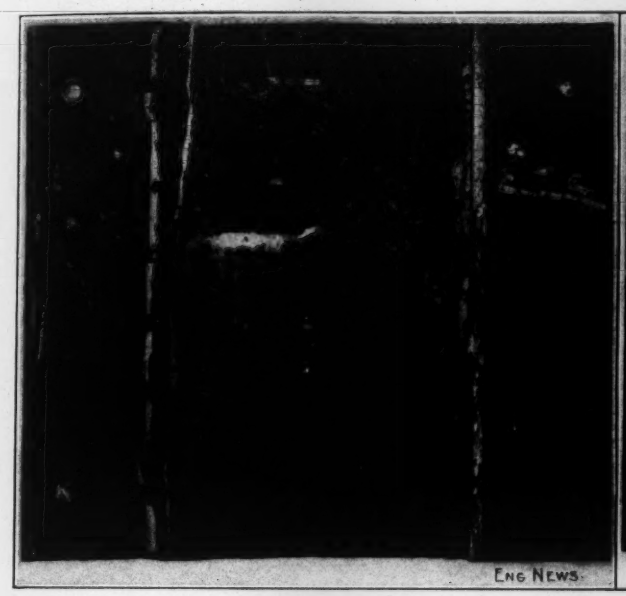


FIG. 4. BOTTOM OF PLATE

(Half of angle remains attached at left and right. Note impressions of channels between gusset plates inside of angles.)

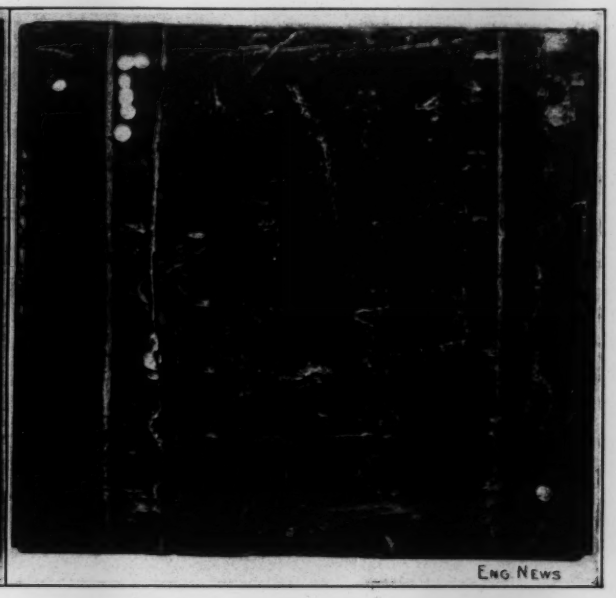


FIG. 5. TOP OF PLATE

(Note impression of web-plates of box-girder, slightly indented in plate.)

FIGS. 4 AND 5. BROKEN CAP-PLATE OF POST 8, GLEN LOCH BRIDGE

We cannot definitely determine whether this rupture was due solely to the superimposed load or some antecedent cause, but there is the evidence of two witnesses who were on the ground that the first indication of the derailment occurred when the train was at a point immediately east of the bridge, where they saw a large amount of fire flying from under the lead engine on train No. 19. This, coupled with the evidence of both enginemen on train No. 19 that they made no application of the air brake at that point, would lead to the conclusion that the train struck an obstruction. This is further borne out by the presence of the mark upon the top of the outer rail, east of Post No. 8, and the marks of the flanges of wheels upon the cross-ties just east of Post N. 8, north of the outer main rail and between the inner main rail and the inner guard rail.

Derailment of the locomotive at this point might have produced by impact a

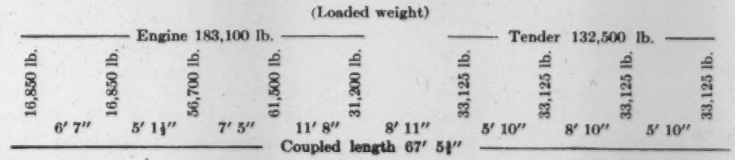
the ends of the girders on it become depressed 18 in. without doing other injury.

POST-CAP 8

A drawing of the post-cap before and after the accident is reproduced from an appendix of the report as our Fig. 3, supplemented with notes on the make-up from the original construction drawings. The cap plate, which had angle brackets

DIAGRAM OF ENGINE AXLE-LOADS.

(Loaded weight)



but no vertical stiffeners to support it, and which carried the load of the girder web-plate 2 in. outside the edge of the column, broke just at the point of support on one side. The box-girder dropped down on the head of the post channels.

A study of the broken plate was made by the railway's engineer of tests, whose results are quoted by the committee to the effect that the plate is normal refined wrought-iron, such as would be accepted today. The test report itself shows, however, that in a bend test the plate performed poorly:

Cold bend on diameter equal to twice thickness of plate,	
With grain, broke at	65°
Across grain, broke at	15°
Hot bend (1700-1800° F.) on diameter equal to thickness of plate,	
With grain, did not break at	180°
Across grain, broke at	100°
Nicked bend	fibrous fracture

In harmony with these results, a poor tensile test was obtained across the grain: tensile strength 30,000 lb. (with grain 47,000), elastic limit 29,000 lb. (with grain 28,500), elongation 4%-6% (with grain 24%), reduction of area 1½-4% (with grain 22%). Thus the plate was relatively brittle across the grain, or was of low resilience.

In the bridge, the plate was placed with grain parallel to the plane of the braced bent. Thus the severe bending stress due to the absence of supporting stiffeners was exerted in the direction in which the plate was brittle.

The composition of the plate was: C 0.033%; Mn 0.025%; P 0.252%; Si 0.115%; S 0.066%.

The regular inspector of bridges reported that he never noticed anything wrong with the plate. On the other hand, the engineer-of-tests' report mentions a fact not noted in the above-printed committee report, namely that the fracture faces show a large area of old break and only a narrow edge of new break. The photograph, Fig. 6, from the report of the engineer of tests, illustrates this.

The same source gives also the photographs reproduced in Figs. 4 and 5, the upper and lower faces of the plate. The webs of the box-girder had indented themselves in the top of the plate, while at the bottom the imprint of the ends of the post channels is visible (probably little more than discoloration).

The facts concerning the post-cap detail indicate that it was long overstrained, and that the cap-plate had cracked on the line of highest bending stress, forming a crack which grew deeper until the plate broke.

The Bridge

The drawing, Fig. 1 (as well as the other of the original shop and erection drawings), fails to show any lateral bracing in the bridge. The structure, obviously of unusual type, would be effectively braced by the floor I-beams were these anchored at the ends. Those framing into the girders are attached by riveted angle clips, but those resting on the abutments appear to have simple masonry-plates fastened to the beams with countersunk rivets but not bolted to the stonework. Therefore, the centrifugal effect of railway traffic on the 2° curve is resisted mainly by the anchor bolts at the abutment ends of the plate girders. The testimony before the committee, accompanying its report as an appendix, informs us that some of these bolts had been broken off for some time.

Further study of this bridge would doubtless be of great value in connection with a full statement of the condition of all parts of the bridge after the wreck. The committee report does not enter upon this field.

The railway committee was appointed by S. C. Long, General Manager, Philadelphia, Penn., and reported to him. It consisted of G. W. Creighton (chairman), General Superintendent Eastern Pennsylvania Division, Altoona; H. C. Bixler, Assistant Division Superintendent, West Philadelphia; J. P. Yorgy, General Car Inspector Eastern Pennsylvania Division, Altoona; J. R. Alexander, General Road Foreman of Engines, Altoona; J. M. Henry, Master Mechanic, Philadelphia; and J. W. Stone, Acting Division Engineer, Philadelphia.

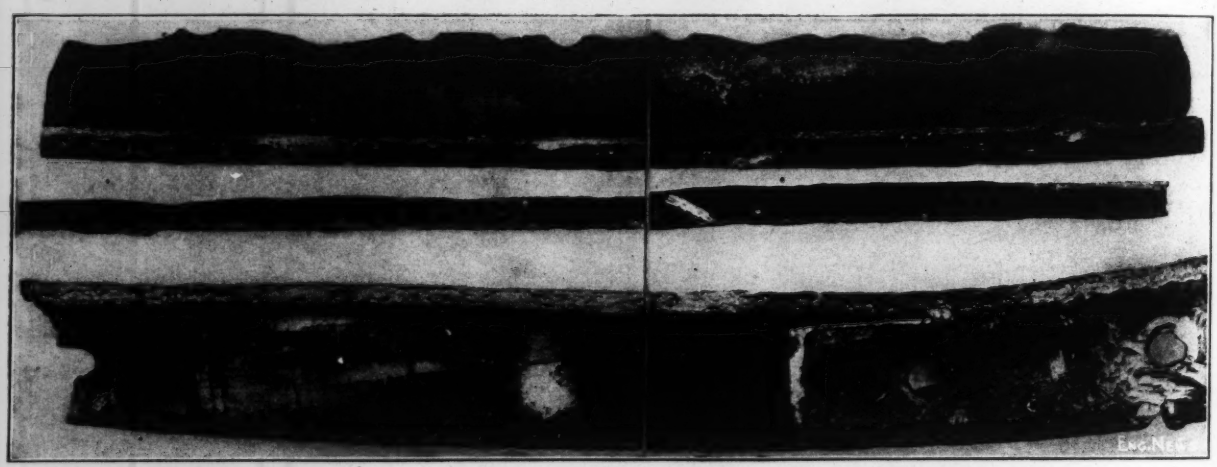


Fig. 6. Edges of Break in Cap-Plate, and Edge of Break in Angle

(Showing progressive fracture. Dark part is old break, light part along bottom edge is final break.)

The Chicago Water-works System, at the end of 1911, included six intake cribs, nine large and five small pumping stations and 49.42 miles of tunnels ranging from 5 to 14 ft. diameter. There were 2362 miles of mains, 20,488 valves and 258,860 services; the number of meters was 16,628, or 6% of the total number of services. Of the six intake cribs in Lake Michigan, one is 4 miles from shore, two are 3 miles distant, two are 2½ miles and one is 2 miles distant. The new Southwest crib and tunnel intake was completed at a cost of $3,036,500; work was commenced in 1906, and water was admitted on Dec. 21, 1911. Its lake tunnel (from the crib) has a daily capacity of 400,000,000 gal.

Drinking Water on Railway Trains continues to be the subject of state action. The State Board of Health of Kansas has issued an order providing that after July 1, 1913, ice must not come in contact with drinking water as supplied on trains or provided in railway stations. It is reported that Dr. F. J. Crumbine, Secretary of the Kansas State Board of Health, has ascertained by tests that the water in railway drinking tanks is contaminated through the handling of the ice put into them.—In New Jersey the State Supreme Court has declared reasonable an order issued by the State Public Utilities Commission to the effect that all railroads which supply drinking water to their patrons must provide either sanitary glasses or individual drinking cups. What is meant by sanitary glasses is not stated in the information at hand.

Failure of Steel-Frame Roof of a Small Theater

The steel-frame roof of the small one-story fireproof Home Theater building on Milwaukee Ave., Chicago, collapsed early in the morning of Dec. 9, the entire width of the roof falling to the floor. At that time the place was empty, so that there were no casualties, but had the accident occurred on Sunday evening, while the theater was open, probably there would have been great loss of life. The place had been reported as dangerous, and an order issued on Saturday by the Building Department requiring it to be closed, but it appears that for some reason the order did not reach the police authorities until after the accident had occurred on Monday.

An investigation of the ruins was made by F. E. Davidson,* M. Am. Soc. C. E., and we are indebted to him for photographs and for the following particulars of the construction and its failure. The structural framework of the roof consisted of 24-in., 80-lb. I-beams girders, spaced 16 ft. 4 in. c. to c., and having a clear span of 48 ft. The roof beams were 12 in., 31½-in. I-beams, spaced 8 ft. c. to c., and framed against the webs of the 24-in. beams. Each 48-ft. girder was in two pieces, spliced at the middle, as shown in the accompanying cut. Each flange was reinforced by one cover-plate extending the full length (but spliced at two points) and one shorter cover-plate. The roof construction was of 12-in. hollow-tile flat arches supported on the 12-in. I-beams, and on top of the tile was approximately 20 in. of cinder concrete with composition roofing.

The failure occurred in the 48-ft. girders. In Fig. 1, one of these is shown shown in Fig. 2. The bill of material for the girders which failed is as follows, this information being secured from the contractor's detail drawings and checked by the inspection made after the accident:

still in place, with the angle-iron connections for the 12-in. I-beams sheared off. The construction of the girders is

- 2 I-beams, 24 in., 80 lb., 25 ft. long.
- 2 Web splice plates, 20x⅜ in., 4 ft. long.
- 2 Cover plates, 8x$\frac{5}{16}$ in., 25 ft. long.
- 4 Cover plates, 8x$\frac{5}{16}$ in., 12 ft. 6 in. long.
- 2 Cover plates, 8x$\frac{5}{16}$ in., 30 ft. long.
- 2 Plates, 9x¾ in., 6 ft. long.

Based upon the actual dead loads which were placed on the beams, it was found that the stresses developed in tension in the bottom flanges of the beams at the splice were considerably in excess of 44,000 lb. per sq.in., or much in excess of the elastic limit of the steel. An inspection of the ruins showed that the girders failed by rupturing of the bottom flange near the center splice, the webs of the girders which failed being broken entirely through, and the top flange and cover plates being buckled.

The architect who designed and superintended the construction of the building was David Saul Klafter. The owner of the building was R. Goldstein. In Illinois, architects are required to be licensed by a state board, and the Business Association of Chicago Architects has filed charges of criminal recklessness with this board, asking that Mr. Klafter's license be withdrawn. A special state's attorney has been assigned to the case, which will be heard in January.

*Patterson & Davidson, Civil and Industrial Engineers, Monadnock Block, Chicago.

FIG. 1. FAILURE OF THE STEEL-FRAME ROOF OF A SMALL THEATER BUILDING IN CHICAGO

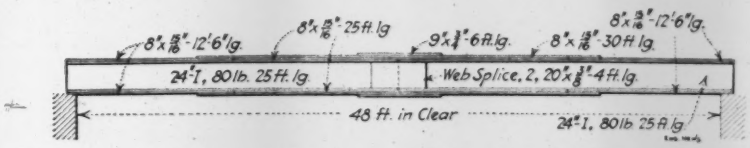


FIG. 2. CONSTRUCTION OF THE ROOF GIRDERS WHICH COLLAPSED

Use of Oil Fuel in the United States Navy*

BY ENGINEER-IN-CHIEF H. I. CONE, U. S. N.

The advantages of oil over coal as a fuel for naval vessels are:

An evaporation per pound of fuel in the ratio of about 14 to 9, and per square foot of heating surface in about the ratio of 10 to 8.

Fuel can be taken aboard more rapidly, without manual labor, and without interruption to the routine of the ship. The problem of fueling at sea is solved.

Steam for full power can be maintained as readily as for lower power. A vessel burning oil is capable of runs at full speed limited in duration only by the supply of the fuel. There is no reduction in speed due to dirty fires or to difficulty in trimming coal from remote bunkers, or to exhaustion of the fireroom force.

There are no cinders and the amount of smoke can be controlled.

A considerable reduction in personnel is possible.

The weight and space required for boilers is reduced: First by the reduction in heating surface required, and second, by the shortening of fire rooms. Consequent on the reduction in heating surface is a decrease in weight and cost of boilers.

Coal and ash-handling gear is eliminated. This renders unnecessary the piercing of the hull for coal trunks and discharges from the ash expellers or ash ejectors.

The stowage and handling of oil is much easier than of coal and will result in a much cleaner ship, with consequent increase in time available for drills.

The mechanical supply of fuel to the boilers gives a prompt and delicate control of the steam supply, permitting more sudden changes in speed than with coal, which is a tactical advantage.

The nature of fuel oil permits utilization of remote portions of the ship and of constricted spaces for its stowage.

*From a paper at the Eighth International Congress of Applied Chemistry.

These advantages have long been recognized by the Navy and there have been experiments with liquid fuel dating back as far as 1867. All these experiments have confirmed our belief in the considerable military advantages which will accrue from its use; but until recently it has been impracticable to use it extensively on account of the uncertainty as to the adequacy of its supply and the sufficiency of its distribution among the seaports of the world. We are now assured, however, as regards the supply, that there is sufficient oil on the public lands of the State of California alone to supply all probable naval demands for one hundred years should oil be burned to the exclusion of coal, and, of course, there is considerable oil in other portions of the U. S. territory. The question as to the distribution of oil among the ports from which fuel might be required by our vessels in time of war is one that is well within our power to solve, as from its nature the oil can be transported and stored more easily than can coal. Indeed for the transport of oil in time of war we are already better provided than for coal, there being a large number of tank steamers flying the American flag.

Oil is therefore certain rapidly to replace coal as a fuel for naval purposes. Since 1907 all torpedo-boat destroyers contracted for, of which there are 29, burn oil exclusively, and the battleships "Delaware," "North Dakota," "Florida," "Utah," "Wyoming," "Arkansas," "Texas" and "New York," contracted for during this period, are fitted to burn oil as auxiliary to coal, each of these vessels carrying about 400 tons of the liquid fuel, to be burned at full power after the coal fires become dirty, or when it becomes difficult to trim coal from the bunkers into the firerooms. In the case of these battleships the advantages of the oil have so appealed to the personnel that oil alone is burned to a great extent in port, and to some extent while cruising, although the installation of the oil-burning equipment did not contemplate these uses.

The "Nevada" and "Oklahoma," the two battleships which have recently been contracted for, will burn oil exclusively. This is perhaps the most radical development in naval engineering since the advent of the turbine. It has permitted in the case of these vessels a reduction in boiler weights, which has made possible the use of heavier armor than has hitherto been employed. The reduction in the length of boiler compartments has permitted the grouping of all boilers under one smoke pipe, which, of course, clears the upper deck considerably and permits more extensive arcs of fire for the turrets.

Aside from the use of oil as fuel under steam boilers, it now seems probable that within comparatively few years oil used in internal-combustion engines will furnish the principal fuel for all naval vessels. This is in consequence of the recent remarkable development of heavy-oil engines of the Diesel type in Europe. Hitherto oil engines have not merited much consideration for large naval vessels on account of the limited power that could be developed in a single cylinder. An installation of any considerable power required a multiplicity of cylinders. Now, however, we are credibly informed that 1000 hp. has been developed in a cylinder about 33 in. in diameter with a 40-in. stroke, at 150 r.p.m. in a 2-cycle marine-type readily reversible engine. This engine has a speed control that is satisfactory, and an economy of fuel consumption probably twice that of a steam engine.

In the U. S. Navy, heavy-oil engines built or so far projected are limited to a number of submarine vessels and to mother ships for submarines. Gasoline is used as fuel for all of our earlier submarines and for a large number of small power boats carried by warships. Its use is likely to be discontinued entirely as soon as suitable heavy-oil motors for the small power boats are developed.

A Turbine Sewer-Cleaning Machine

A sewer-cleaning machine, which operates somewhat on the principle of the "go-devil" used in cleaning water mains (except that it is not self-propelling),

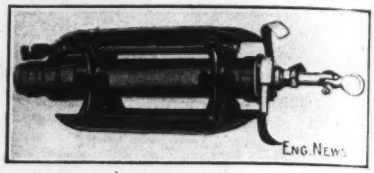


FIG. 1. THE TURBINE SEWER CLEANER

has been used with success in several cities during the past two years, and has been found capable of cleaning sewers when almost entirely choked with deposit. The machine itself, shown in Fig. 1, consists of a central brass barrel about 25 in. long, with arms carrying four guides, and at the forward end is a miniature hydraulic turbine with three revolving scraper blades. For ordinary work, a rope or cable is passed from one manhole to another by means of quick-

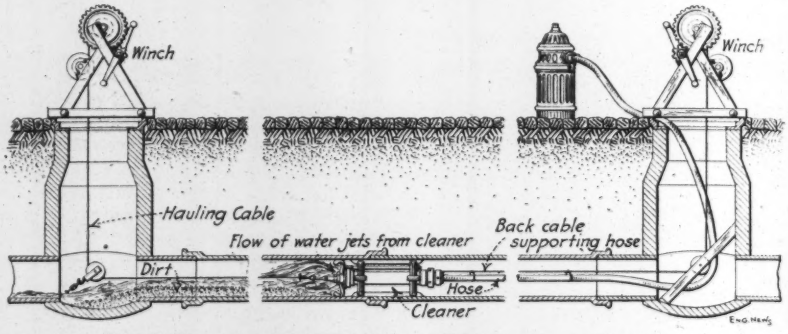


FIG. 2. METHOD OF OPERATING THE TURBINE SEWER CLEANER

FIG. 3. METHOD OF FORCING THE TURBINE SEWER CLEANER THROUGH A CLOGGED SEWER

coupling sewer rods, the far end of the cable being then attached to a swivel clevis or shackle at the front end of the cleaner. A fire-hose, attached to a hydrant, is coupled to the rear of the barrel. Another cable is also connected to the rear of the machine, and this is secured to the hose at intervals by chain clamps in order to relieve the strain on the hose. Each cable passes over a guide pulley at the bottom of the manhole, and has its upper end wound upon the drum of a compound-geared winch or hand-windlass, which is erected over each manhole.

The apparatus being thus connected, the cleaner is entered into the sewer at the upper manhole and the water turned on, while the windlass at the next manhole is operated so as to pull the cleaner through the sewer. This arrangement is shown clearly in Fig. 2. With a water pressure of 60 lb., the turbine is said to develop about 3 hp. The revolving blades cut and churn all obstructions and with the strong jets of water the material is churned into a grout which flows away in front of the machine to the lower manhole. The machine can be hauled back with the rear cable for a second trip if necessary.

When a sewer is so choked that the pulling cable cannot be passed through by the sewer rods, the cleaner is pushed instead of pulled along the sewer. For this purpose the head rod is attached to the rear end of the cleaner, and a special worm-gear forcing jack is placed at the bottom of the manhole. This manhole would be at the lower end of the sewer grade. The hose is attached to the barrel as before, and the machine is allowed to work a few minutes to cut and soften the material before being forced ahead. Fig. 3 shows the cleaner attached to the rod, the rear of which is fitted to the forcing jack, at the left.

At Kansas City, a report of the Superintendent of Street Repairs shows that in 14 days 7800 ft. of sewer were cleaned in this way at a cost of 3.15c. per ft., while the old method cost from 15 to 40c. per ft. At Milwaukee, in April, 1912, the Superintendent of Sewers (H. J. Kruse) stated that 320 ft. of a 12-in. sewer was cleaned in 16 hr. with four men at a cost of 5c. per ft. This sewer was so full of grease from a factory that it was hard work to get the rods through, and without the machine it would have been necessary to dig up the sewer and relay it, at a cost of probably $3 to $4 per ft. (the frost being 6 ft. deep in the ground at the time). The City Engineer of Memphis, Tenn. (J. H. Weatherford), stated that a 15-in. sewer about 225 ft. long, which was almost completely stopped, was cleaned in about 15 min. after the rods were put through.

The device described is manufactured by the Turbine Sewer Renovating Co., 197 11th St., Milwaukee, Wis.

Alfred Pancoast Boller*

The death of Alfred Pancoast Boller, Vice-President of the American Society of Civil Engineers, and President of the American Institute of Consulting Engineers, on Dec. 9, has removed from the profession of civil engineering one of its great leaders.

Mr. Boller was born in Philadelphia, Penn., in 1840. He was graduated from the University of Pennsylvania with the degree of A. M. in 1858, and from the Rensselaer Polytechnic Institute in 1861, with the degree of C. E.

After leaving Rensselaer he engaged actively in his chosen profession; commencing as a rodman on the Nesquehoning Valley R.R., he soon became an assistant engineer and topographer. In this latter capacity he made an elaborate topographical map of the Middle and Southern anthracite coal fields, with their various outlets to market.

Alf. P Boller

PHOTO BY OLIVER LIPPINCOTT, N. Y. CITY

In June, 1862, he was detailed as an assistant on the repairs of the Lehigh Canal, which had been damaged almost to obliteration by a great freshet. In the spring of 1863 he became a member of the staff of the General Manager of the Philadelphia & Erie R.R., which had just been leased by the Pennsylvania R.R. Two years later he was appointed Engineer of Bridges of the Atlantic & Great Western R.R.; he planned an International Bridge over the Niagara River at Black Rock and the Catarraugus viaduct, works never carried out due to the collapse of the railroad company in the same year.

In the fall of 1865, Mr. Boller entered the service of the Hudson River R.R. as Chief Engineer, which position he resigned a short time later to enter the iron business with Samuel Milliken (Milliken & Boller) as agents of the Phœnix Iron Co., in New York and New England. In 1870 he became Vice-President and Engineer of the Phillipsburg Manufacturing Co., until the panic of 1873 caused the failure of that concern. During its existence numerous railroad and highway bridges were built under the supervision of Mr. Boller. After the failure of this concern, Mr. Boller became Chief Engineer of the Manhattan Elevated R.R., New York City, and had charge of the construction of its elevated structure; later he became Chief Engineer of the West Side & Yonkers R.R and designed its bridge over the Harlem River at Eighth Ave., New York City.

On the completion of this work, Mr. Boller opened an office as Consulting Engineer and at once acquired a large and important practice. He designed and built many of the most notable bridges in the country, including the bridge over the Harlem River at Madison Ave., New York City; the Albany & Greenbush bridge over the Hudson River at Albany, N. Y.; the Arthur Kill bridge, New York; the bridge over the Thames River at New London, Conn.; the viaduct and bridge over the Harlem River at 155th St. and Seventh Ave., New York City, and the bridge over the St. Louis River at Duluth, Minn.

In 1898, Mr. Boller formed a partnership with Henry W. Hodge, under the firm name of Boller & Hodge, and the firm has since designed and constructed many large structures, including the bridge over the Arkansas River at Little Rock, Ark., various bridges for the City of New York, the cantilever bridges for the Wabash R.R. across the Monongahela River at Pittsburgh, Penn., and across the Ohio River at Mingo Junction, the Montreal River viaduct for the Algoma Central R.R., and the bridge over the Connecticut River at Saybrook for the State of Connecticut. The firm was also the consulting engineer for the steel construction of the Singer Building and the Metropolitan Life Building towers in New York City. In 1912 the firm was enlarged by the addition of Howard C. Baird, and the firm of Boller, Hodge & Baird is now constructing the bridge over the Mississippi River at St. Louis, Mo., for the City of St. Louis; the bridge over the Connecticut River at East Haddam, for the State of Connecticut; the concrete bridge over the Hillsborough River, for the City of Tampa, Fla., and many railroad bridges.

Mr. Boller, or his firm, has been frequently retained as consulting engineer

*Contributed by Henry Wilson Hodge.

by the United States Government, the State of New York, and the City of New York, as well as by many other cities and railway companies, including the National Rys. of Mexico and a large number of foreign railway corporations. The firm has designed bridges for railroads in various parts of South and Central America, Cuba, the Philippines, and Hayti.

Mr. Boller was not only an expert in the matter of steel construction, but he was also an eminent expert in the matter of foundations, and successfully constructed some of the most difficult foundations in the country.

He was a leader in the science of iron and steel bridge construction from its beginning, and he had the skill and daring to design each successive structure right up to the possibilities of construction, so that he had a large part in bringing the art of bridge design and manufacture up to the high state it has now reached. The structures he built will form lasting monuments to him, as they are in many cases the greatest of their type so far built.

It was not only as an engineer that Mr. Boller won the admiration of all who knew him, but by the many fine characteristics which he possessed, and by his high ideals of life. He was always willing to give his time for the benefit of his fellow men. He always remembered that an engineer is the arbiter between two parties to a contract, and saw that the contractor received just as fair treatment as the owner. He was active in church and philanthropic work, and was looked up to for advice and helpful assistance in the community.

He passed away after an illness of nearly a year, having completed nearly 73 years of able, helpful work. He was both an engineer and a gentleman of the highest type, and all who knew him will earnestly join in the commendation, "Well done, good and faithful servant."

George Albert Kimball*

A brief note in our issue of last week announced the sudden death, on Dec. 3, at his home in Boston, Mass., of George Albert Kimball, Chief Engineer of subway and elevated construction of the Boston Elevated Ry. The previous Friday he had been in the best of health and had been about his work as usual. The immediate cause of his death was heart failure induced by an attack of acute indigestion.

Mr. Kimball was born in Littleton, Mass., May 14, 1850. His early education was obtained in the public schools of Littleton and at Appleton Academy, New Ipswich, N. H. He prepared to enter Dartmouth College and the Thayer School of Civil Engineering, but was compelled to relinquish his studies on account of eye trouble.

In 1869 he entered the office of Frost Bros., Somerville, Mass., and was engaged in surveying and engineering work for this firm until 1874, when he opened an office and began business for himself. Two years later he was elected City Engineer of Somerville, and continued to hold this office under different administrations for eleven years. During seven years of this period he also served as a member of the Somerville board of health. In 1887 he resigned and opened a private office in Boston, Mass.

His ability soon brought him into prominence and in the following year he was appointed a member of the Massachusetts Grade Crossing Commission,

Yours Truly
George A. Kimball

which conducted an extensive investigation and reported upon all the grade crossings in the state. This report has been the basis of a large part of the work done since that time in abolishing these crossings. Another important state engineering commission of which Mr. Kimball was for many years a member is the Metropolitan Sewerage Commission. In his work at Somerville, he had early established a reputation as an expert on sewerage and sewage disposal, and, although his later life was largely spent in entirely different work, he remained until his death the consulting engineer of the cities and towns of Milles, Malden, Brockton, Milton, New Bedford, Winthrop and Peabody, Mass., and Montpelier, Vt.

In 1896 he was appointed Chief Engineer of elevated and subway construction of the Boston Elevated Ry. Co. and held this position continuously until his death. In this capacity Mr. Kimball passed upon and was responsible for the plans for the Tremont St. subway, the Washington St. tunnel, the East Boston tunnel and the Charleston bridge, as well as the subway in Boylston St., which is now in the progress of construction. He designed and superintended the construction of the elevated structure from Forest Hills to Sullivan Square, the Cambridge subway, and the extension of the elevated structure to Malden so far as built. That the company might have the benefit of experience in rapid-transit facilities abroad, Mr. Kimball was detailed in 1902 to go to Europe and study the methods of transportation in the great cities of England, Scotland, France, Germany and Austria.

Mr. Kimball joined the American Society of Civil Engineers as a Junior in 1875, and he was elected a member in 1891. In 1910 he was elected one of the directors of the society. He was an active member and a past-president of the Boston Society of Civil Engineers; he was also a member of the New England Water Works Association. In 1910 he was elected a member of the Institution of Civil Engineers of Great Britain. He was also a member of the American Institute of Consulting Engineers.

Mr. Kimball was married in 1872 to Lizzie E. Robbins, who, with four children, survives him.

*From data gathered by F. I. Winslow, Public Works Dept., Boston, Mass.

The Italian Steamer "Derna" was Struck by the new British dreadnaught "Centurion," in the English Channel, on Dec. 10, and sank immediately, carrying down a crew of 25 men. The battleship was steaming at an estimated speed of 20 knots an hour before dawn, when the other boat tried to cross her bows.

A Butting Collision took place, on Dec. 7, at Pen-Mar, Penn., between a fast freight and an empty passenger train on the Western Maryland Ry. According to press reports the accident, which resulted in five deaths and the injury of six other persons, was due to failure to obey a train order.

A Train Derailment, on a branch of the Baltimore & Ohio R.R., near Glencoe, Penn., resulted in the death of five men, on Dec. 12. Two engines and 42 cars loaded with coal got beyond control of the air brakes and ran away, traveling about eight miles before a sharp curve caused the derailment.

A Fire, in Cincinnati, Ohio, on Dec. 11, consumed a large part of a city block and did damage estimated at from $700,000 to $1,500,000. Six lives were lost. The fire spread to the Union Trust Building of 17 stories, one of the oldest sky-scrapers in the city, and to adjoining business buildings.

A 14-in. Gun Exploded at the Sandy Hook testing grounds, on Dec. 9, endangering the lives of the test party. It was the last one of four similar guns, built for the Panama Canal fortifica-

tions. It had gone through the preliminary tests with light charges and was being tried with the full charge of 350 lb. of smokeless powder when the explosion occurred. According to newspaper reports, the muzzle of the gun was torn off and thrown some distance, and the breech-block was thrown a quarter of a mile back of the gun. The members of the test party were in a bombproof shelter when the gun was fired, in accordance with regulations, and thereby escaped injury.

A 13½-in. gun, being tested for the British Government, at Shoeburyness, exploded during the early part of December, but the accident in that case was attributed, in the English press, to a premature explosion of the charge in the projectile before it had left the rifle barrel.

Panama Canal Excavation for November totaled 2,505,358 cu.yd., a daily average of 104,390 cu.yd. for each of the 24 working days. The rainfall for the month was 10.70 in. The amount of concrete laid in November was 55,078 cu.yd. and 248,630 cu.yd. of fill were placed in dams.

The Sanitary Inspection of Railway Trains and of Passenger Vessels is to be made in the future by the officials of the U. S. Public Health Service. As we understand it, lacking a specific appropriation for the purpose the various members of the Service will conduct inspections while they are aboard railway trains or vessels.—At Chicago, Dr. George D. Young, Commissioner of Health, has requested a number of steamship companies to confer with him regarding the discharge of sewage from vessels into the harbor and lake at points which would endanger the water-supply drawn from Lake Michigan.

A Very Large Reinforced-concrete Column was tested in the Pittsburgh laboratory of the U. S. Bureau of Standards, on Dec. 14, 1912, before a delegation from the National Association of Cement Users, meeting that week in Pittsburgh. It is reported to be the largest reinforced-concrete column ever loaded to destruction. It was a hooped column 16 ft. long, 30 in. in diameter with a core diameter of 27 in. giving a total core area of 572.55 sq.in. The reinforcement consisted of seven longitudinal 1⅛-in. round rods, a ½-in. wire helix with 3-in. pitch and 31 flat spirals of No. 5 wire of 3 turns spaced vertically 6 in. center to center. The column commenced to show signs of distress about 1500 lb. per sq.in., core area, and failed at a total load of 1,800,000 lb., or about 3100 lb. per sq.in., core area.

Inventors of Automatic Train Stops are to receive some encouragement as a result of the two recent crossover accidents on the New York, New Haven & Hartford R.R. In the New York "Times," of Dec. 14, this company makes the following offer:

A REWARD OF $10,000

to be paid on the order of the Interstate Commerce Commissioners and the Railroad Commissioners of Massachusetts and the Public Utilities Commissioners of Connecticut, to whomsoever shall first invent an automatic device that will safely arrest an express steam locomotive that has passed danger signals; the test of efficiency to be its adoption within the year 1913, 1914 or 1915 by the New Haven Road, the New York Central, or the Pennsylvania and its approval or recommendation by the Interstate Commerce Commissioners.

CHARLES S. MELLEN,
President.

Personals

Mr. Fred M. Lockwood, Engineer of the Los Angeles, Calif., sewer department, has been promoted to be Assistant Superintendent of Sewers.

Mr. G. C. Millet, Engineer of the Coast Lines of the Atchison, Topeka & Santa Fé Ry., at Los Angeles, Calif., has been appointed Assistant to the Chief Engineer.

Mr. John T. Hendricks, Vice-President of the Western Maryland Ry., has resigned, effective Dec. 31, to become General Traffic Manager of the Missouri Pacific Ry.

Mr. A. S. Going, M. Am. Soc. C. E., who has been Locating Engineer of the Grand Trunk Ry. System, has been appointed Engineer of Construction, with office at Montreal, Que.

Mr. J. L. Hayes Trainmaster of the Pere Marquette R.R. at Plymouth, Mich., has been promoted to be Superintendent of the Port Huron division, with headquarters at Saginaw, Mich.

Mr. W. K. Tasker, formerly Superintendent of the Pere Marquette R.R., at Saginaw, Mich., has been made Superintendent of the Canadian division with headquarters at St. Thomas, Ont.

Mr. F. Boeken, formerly Superintendent of the Geary St. cable railway, San Francisco, Calif., has been appointed Assistant Superintendent of the Municipal street railway system at a salary of $2100 per annum.

Mr. J. J. Waters has been appointed Superintendent of Motive Power of the Pere Marquette R.R., with headquarters at Grand Rapids, Mich., succeeding Mr. W. L. Kellogg, resigned, as noted in our issue of Nov. 28.

Mr. E. J. Haddock, M. Am. Soc. M. E., has resigned as Chief Engineer of the Montgomery Coal Washing & Manufacturing Co, Birmingham, Ala., to become Chief Engineer of the T. L. Smith Co., and the Smith & Post Co., Milwaukee, Wis.

Rear-Admiral Frank E. Beatty, who has been in command of the Washington (D. C.) Navy Yard and the naval gun factory since 1910, is to be relieved, Jan. 1, by Capt. H. P. Jones, at present in command of the battleship "Rhode Island."

Mr. D. I. Roberts, General Manager of the Quebec, Montreal & Southern Ry., a subsidiary of the Delaware & Hudson R.R., at Montreal, Que., has been elected President of the United States Express Co., succeeding the late Thomas C. Platt, who died in 1910.

Mr. Hibbard A. Paine, Assoc. M. Am. Soc. C. E., for 17 years engaged in railway location and construction work in various parts of the United States, and for the past 4½ years Road Engineer of Caroline County, Md., has resigned to take up private practice in Denton, Md.

Mr. J. W. Buzick, Assistant Engineer of the Conley Frog & Switch Co., Memphis, Tenn., has been appointed Chief Engineer, succeeding Mr. Fred C. Taylor, resigned, to become Manager of the insulation department of the St. Louis, Mo., branch of the H. W. Johns-Manville Co.

Mr. W. F. Schaff, Assistant Superintendent of the Lake Shore & Michigan Southern Ry., at Cleveland, Ohio, has been promoted to be Superintendent at Detroit, Mich., succeeding Mr. T. W. Niles, who has been assigned to special duties under the General Superintendent.

Mr. R. S. Kent, M. Am. Soc. M. E., of Brooklyn, N. Y., has been appointed Designing and Supervising Engineer of the new sugar refinery to be erected at St. John, N. B., by the Atlantic Sugar Refineries, Ltd. This plant will be designed to melt 750,000 lb. of raw sugar daily.

Mr. Charles W. Beaver, for the past two years Special Representative of the Yale & Towne Manufacturing Co., in Europe, and previously Assistant Manager of the company's chain block department has been appointed Manager of this department succeeding Mr. R. T. Hodgkins resigned.

Mr. Robert I. Farrington has resigned as Second Vice-President of the Great Northern Ry. Mr. Farrington began railway work as a clerk in the auditor's office of the Northern Pacific Ry., in 1882. He was appointed Auditor of disbursements of the Great Northern Ry. in 1894 and elected Second Vice-President in 1902.

Mr. Roy D. Hunter has resigned as General Sales Manager of the Sullivan Machinery Co., with headquarters at Chicago, Ill. He is succeeded by Mr. Howard T. Walsh, European Manager of the company at London, England. Mr. Austin Y. Hoy, Western representative at Spokane, Wash., succeeds Mr. Walsh as London.

Mr. James B. Goodwin, M. Am. Soc. C. E., recently Superintendent of Construction and Assistant General Manager of the Mount Hood Railway & Power Co., Portland, Ore., has been appointed Construction Engineer for the City of Edmonton, Alberta, a city which owns practically all its public utilities and proposes to do a large amount of new work in the coming year.

Mr. Charles Lathrop Pack, of Lakewood, N. J., has been elected President of the Fifth National Conservation Congress, to be held in the fall of 1913. Mr. Pack is a native of Cleveland, Ohio, and a former President of the Cleveland Chamber of Commerce. He has been a practical lumberman and has taken an active interest in economic forestry. He is a director of the American Forestry Association.

Mr. George A. Taber, M. Am. Soc. C. E., Engineer in Charge of construction of water mains, Boroughs of Manhattan, the Bronx and Richmond, New York City, has been appointed Consulting Professor of water supply and sewerage at the Brooklyn Polytechnic Institute in charge of the evening courses in these subjects. These courses were formerly conducted by Prof. George C. Whipple, M. Am. Soc. C. E., now head of the department of sanitary engineering at Harvard University.

Mr. V. A. Riton, whose appointment as General Superintendent of the Eastern general division of the Norfolk & Western Ry., was noted in our issue of Dec. 5, began his railway work as a rodman on the construction of the New York & Canada Ry., in 1872. He was successively Levelman and Assistant Engineer of the same railway, and in 1877 he became Assistant Engineer of the Platts-

burg & Dannemora R.R., now a subsidiary of the Delaware & Hudson Co. In 1879 he was Assistant Engineer of the Chicago & Northwestern Ry. for a short time, and then held the same position with the Chicago, Milwaukee & St. Paul Ry. Mr. Riton began his experience in the operating department as Road Master of the Mineral Point division of the Chicago, Milwaukee & St. Paul Ry. in 1882. From January, 1895, to November, 1898, he was Superintendent of the Cascade division of the Great Northern Ry., at Leavenworth, Wash. Since December, 1898, he has been continuously with the Norfolk & Western Ry.

Mr. S. B. Williamson, M. Am. Soc. C. E., has resigned as Division Engineer, Pacific division, Panama Canal, to become Principal Assistant Engineer of J. G. White & Co., Ltd., of London, England, where he will have supervision of construction work in all parts of the world, except in the United States and its possessions. Mr. Williamson was engaged in railway construction work in the northwestern part of the United States from 1886 to 1890, and after two years' general engineering practice in Southern cities, he entered the service of the United States Government on engineering work in connection with the Tennessee River. In 1900 he was transferred to Newport, R. I., as Assistant Engineer in charge of the fortification work in that district. In this work and in his work on the Tennessee River he was closely associated with Col. G. W. Goethals, and when the latter became Chief Engineer of the Isthmian Canal Commission, he sent for Mr. Williamson to become Division Engineer of the Pacific locks and dams. Mr. Williamson has been Division Engineer since May, 1907, and was the only civilian engineer to have charge of a construction division, the other two divisions being in charge of army officers.

Obituary

Addison E. Greene, Superintendent of Construction of the Roebling Construction Co., New York City, died at his home in Brooklyn, Dec. 14, aged 46 years.

John Hawkesworth, Assoc. M. Am. Soc. C. E., of New York City, died Dec. 10. He was born in New York City in 1883, and was graduated from Columbia University in 1904.

William D. Barclay, General Manager of the Canadian Northern Quebec Ry., the Halifax & Southwestern Ry., and other subsidiary lines of the Canadian Northern Ry., died Dec. 13, at Riverside, Calif., where he had gone in search of health. He was 61 years old.

William W. Dey, an old-time railway contractor, who built large sections of what is now the Pennsylvania R.R. system, died at his home in New York City, Dec. 13, aged 81 years. He served in the U. S. Army during the Civil War, and at its close established a contracting business in Perth Amboy, N. J.

Thomas C. J. Baily, Jr., M. Am. Soc. C. E., Engineer of Bridges of the District of Columbia, died on Dec. 7, 1912, in a hospital at Mexico City, Mexico, at which city has was stricken with an attack of malaria on his way back to this country from a professional trip to Mexico. So far as is now known Mr. Baily was alone in the city and his friends and relatives did not know of his illness until immediately before its fatal termination. He was a graduate of Lehigh University, Class of 1890, and after a short service on the Mississippi River improvement work, entered the engineering department of the District of Columbia in 1893, becoming Engineer of Bridges in 1910. He was the author of several papers in the technical press and society proceedings and was also a contributor to the American Civil Engineers Pocket Book.

George Burnham, former President of the Baldwin Locomotive Works, died at his home in Philadelphia, Penn., Dec. 10, in his 96th year. Mr. Burnham was born in Springfield, Mass., Mar. 11, 1817, and at the age of 15 years went to Philadelphia as a grocer's clerk. While working in this capacity he met Matthias W. Baldwin, founder of the famous Baldwin Locomotive Works, of Philadelphia. Entering Mr. Baldwin's employ as a clerk and bookkeeper, Mr. Burnham's position and opportunities expanded as the business grew and after Mr. Baldwin's death he became a member of the firm. In 1873, Mr. Burnham became senior member of the firm of Burnham, Parry, Williams & Co., owners of the works, and so remained until the incorporation of the Baldwin Locomotive Works in 1909, when he retired. He is survived by two sons, George Burnham, Jr., Asoc. Am. Soc. C. E., a director of the Baldwin Locomotive Works, and William Burnham, M. Am. Soc. M. E., President and Treasurer of the Standard Steel Works Co., Philadelphia, Penn., and one daughter, Mary A. Burnham.

F. H. Bainbridge, formerly Bridge Engineer of the Chicago & Northwestern Ry., died at his home in Ravenswood, Ill., Dec. 4. He was a graduate of the Rensselaer Polytechnic Institute, class of 1884, and began his engineering work as an inspector in the shops of the Northern Pacific Ry. For seven years, 1886 to 1893, he was a draftsman with the Edge Moore Bridge Works, at Wilmington, Del. In 1893 he went to Chicago, Ill., as Resident Engineer for the Pencoyd Iron Works and the Mount Vernon Bridge Co. From 1895 to 1898 he was successively Engineer of the Buffalo & Niagara Falls Electric Ry., with Alfred Noble, Civil Engineer, New York City, and with contractors on the Illinois Central and the Atchison, Topeka & Santa Fé Rys. In 1898 he was Engineer in charge of construction work for the Chicago & Northwestern Ry. With the exception of a year as Engineer for the Western Expanded Metal Co. (1902-1903) he was successively Assistant Engineer of Bridges, Bridge Engineer, and Principal Assistant Engineer of the Chicago & Northwestern Ry.

Charles A. Allen, M. Am. Soc. C. E., Consulting Engineer, died at his home in Worcester, Mass., Dec. 9. He was born in Worcester in 1852. His education was obtained in the public schools and at the Worcester Academy. After completing his studies he entered the employ of Phineas Ball, an early hydraulic engineer, and there laid the foundations of his own reputation as a hydraulic engineer. After a period as Engineer of the Worcester & Nashua R.R., now a subsidiary of the Boston & Maine R.R., he opened an office in Worcester for private practice. Some years after he was appointed City Engineer of Worcester and served in this capacity for 15 years, during a period of great development in which the sewage-disposal plant at Quinsigamond was laid out and extensive additions made to the water-supply and sewerage systems. To perfect plans for the sewage-disposal works Mr. Allen was sent to Europe to study European methods, and the Worcester plant built under his administration was the first large sewage treatment plant constructed in this country. After resigning as City Engineer he was actively engaged in consultation work until his last illness. He was also a member of the Boston Society of Civil Engineers.

Engineering Societies

COMING MEETINGS

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS.
Dec. 26-28. Annual meeting at Chicago, Ill. Secy., C. O. Reed, Urbana, Ill.

AMERICAN ASSOCIATION FOR THE ADVANCEMENT OF SCIENCE.
Dec. 30-Jan. 6. Annual meeting at Cleveland, Ohio. Secy., L. O. Howard, Smithsonian Institution, Washington, D. C.

MINING AND METALLURGICAL SOCIETY OF AMERICA.
Jan. 14. Annual meeting at New York City. Secy., W. R. Ingalls, 505 Pearl St., New York.

AMERICAN WOOD PRESERVERS' ASSOCIATION.
Jan. 21-23. Annual convention at Chicago, Ill. Secy., F. J. Angier, B. & O. RR., Baltimore, Md.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 21-23. Annual meeting at New York City. Secy., W. W. Macon, 29 W. 39th St., New York.

American Society for the Advancement of Science—The annual meeting of the Association will take place in Cleveland, Ohio, lasting from Dec. 30 to Jan. 6. At the meetings of Section D on Engineering, Jan. 3-4, there will be presented 26 papers on the subject of highway engineering.

American Mining Congress—The following officers for 1913, were elected at the annual meeting in Spokane, Wash., Nov. 25-28: President, D. W. Brunton; Vice-Presidents, Hennon Jennings, E. A. Montgomery and Carl Scholz; Directors (3 years), D. W. Brunton, W. C. Conrad, G. H. Dern and H. L. Day; (1 year), W. B. Shackleford; Secretary, J. F. Callbreath.

American Electric Railway Association—The annual mid-year meeting will be held from Wed., Jan. 29, to Sat., Feb. 1, inclusive. Previously these meetings in the middle of the year have been executive sessions and only official representatives of member companies have been admitted. In accordance with action taken a year ago all meetings are now open to anyone connected with the Association in any way, whether officially representing a railway company or not.

The following program has been announced for the coming meetings: Wed., Jan. 29, and Thurs., Jan. 30, meetings of various committees of the Association and affiliated associations (Engineering, Accounting, Transportation and Traffic, and Claim Agents) at the Engineering Societies Building, New York City; Fri., Jan. 31, morning and afternoon sessions of the American Association, annual banquet extended to the Association by the American Electric Railway Manufacturers Association, Hotel Astor; Sat., Feb. 1, meetings of various committees of the Association and affiliated associations.